NATIONAL ASSOCIATION

OF

CEMENT USERS

PROCEEDINGS

OF THE

EIGHTH ANNUAL CONVENTION

Held at Kansas City, Mo.,

March 11, 12, 13, 14, 15, 16, 1912

VOLUME VIII

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The Association is not responsible, as a body, for the statements and opinions advanced in its publications.

Transport

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LIST OF OFFICERS

OF

THE NATIONAL ASSOCIATION OF CEMENT USERS

1912.

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SECOND VICE-PRESIDENT.

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EDWARD S. LARNED.

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IRA H. WOOLSON.

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HENRY C. TURNER.

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(See page 7.)

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J. B. LANDFIELD,
C. R. MILLER,
A. E. SNODGRASS.

SPECIFICATIONS AND METHODS OF TESTS FOR CONCRETE MATERIALS.

SANFORD E. THOMPSON, *Chairman*.

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WILLIAM B. FULLER, ARTHUR N. TALBOT.

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Arkansas—MARTIN NELSON.
California—LEROY ANDERSON.
Colorado—ALVIN KEYSER.
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Florida—J. J. VERNON.
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Utah—F. S. HARRIS.
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Washington—O. L. WALLER.
West Virginia—I. S. COOK.
Wisconsin—CHARLES A. OCOCK.
Wyoming—J. C. FITTERER.

PAST OFFICERS.

President.	1905	John P. Given. (Presiding Officer First Convention.)
	1905–11	Richard L. Humphrey.
First Vice-President.	1905	A. L. Goetzmann.
	1906–9	Merrill Watson.
	1909–11	Edward D. Boyer.
Second Vice-President.	1905–6	John H. Fellows.
	1907–10	M. S. Daniels.
	1911	Arthur N. Talbot.
Third Vice-President.	1905	H. C. Quinn.
	1906–7	O. U. Miracle.
	1908	S. B. Newberry.
	1909–11	E. S. Larned.
Fourth Vice-President.	1905–7	A. Monsted.
	1908–9	George C. Walters.
	1909–10	F. A. Norris.
	1911	Ira H. Woolsen.
Treasurer.	1905	A. S. J. Gammon.
	1906–11	H. C. Turner.
Secretary.	1905–6	Charles C. Brown.
	1907	W. W. Curtis.
	1908–9	George C. Wright.
	1910	Edward E. Krauss (Acting)
	1911	Edward E. Krauss.

CHARTER

OF

THE NATIONAL ASSOCIATION OF CEMENT USERS.

KNOW ALL MEN BY THESE PRESENTS, That we, the undersigned, all of whom are citizens of the United States, and a majority of whom are residents of the District of Columbia, have associated ourselves together for the purpose hereinafter set forth and desiring that we may be incorporated as an Association under sub-chapter three (3) of the Incorporation Laws of the District of Columbia, as provided in the Code of Law of the District of Columbia, enacted by Congress and approved by the President of the United States, do hereby certify:

1. Name. The name of the proposed corporation is "The National Association of Cement Users."

2. Term of Existence. The existence of the said corporation shall be perpetual.

3. Objects. The particular business and objects of the said corporation shall be to disseminate information and experience upon and to promote the best methods to be employed in the various uses of cement by means of convention, the reading and discussion of papers upon materials of a cement nature and their uses, by social and friendly intercourse at such conventions, the exhibition and study of materials, machinery and methods and to circulate among its members by means of publications the information thus obtained.

4. Incorporators. The number of its managers for the first year shall be fifteen.

In Witness Whereof, we have hereunto set our hands and seals this fourteenth day of December, A. D. 1906.

RICHARD L. HUMPHREY, (SEAL)
JOHN STEPHEN SEWELL, (SEAL)
S. S. VOORHEES. (SEAL)

OFFICE OF RECORDER OF DEEDS,
DISTRICT OF COLUMBIA.

This is to certify that the foregoing is a true and verified copy of a Certificate of Incorporation, and of the whole of such Certificate as received for record in this office at 9:49 A. M., the 19th day of December, A. D. 1906.

In testimony whereof I have hereunto set my hand and affixed the seal of this office, this 20th day of December, A. D. 1906.

(Signed) R. W. DUTTON,
Deputy Recorder of Deeds,
District of Columbia.

(SEAL)

BY-LAWS.

ARTICLE I.

MEMBERS.

SECTION 1. Any person engaged in the construction or maintenance of work in which cement is used, or qualified by business relations or practical experience to co-operate in the purposes of this Association, or engaged in the manufacture or sale of machinery or supplies for cement users, or a man who has attained eminence in the field of engineering, architecture or applied science, is eligible for membership.

SEC. 2. A firm or company shall be treated as a single member.

SEC. 3. Any member contributing annually twenty or more dollars in addition to the regular dues shall be designated and listed as a Contributing Member.

SEC. 4. Application for membership shall be made to the Secretary on a form prescribed by the Board of Direction. The Secretary shall submit monthly or oftener if necessary to each member of the Board of Direction for letter ballot a list of all applicants for membership on hand at that time with a statement of the qualifications, and a two-thirds majority of the members of the Board shall be necessary to an election.

Applicants for membership shall be qualified upon notification of election by the Secretary by the payment of the annual dues, and unless these dues are paid within 60 days thereafter the election shall become void. An extract of the By-Laws relating to dues shall accompany the notice of election.

SEC. 5. Resignations from membership must be presented in writing to the Secretary on or before the close of the fiscal year and shall be acceptable provided the dues are paid for that year.

ARTICLE II.

OFFICERS.

SECTION 1. The officers shall be the President, two Vice-Presidents, six Directors (one from each geographical district), the Secretary and the Treasurer, who, with the five latest living Past-Presidents, who continue to be members, shall constitute the Board of Direction.

SEC. 2. The Board of Direction shall, from time to time, divide the territory occupied by the membership into six geographical districts, to be designated by numbers.

SEC. 3. The terms of office of the President, Secretary and Treasurer shall be one year; of the Vice-Presidents and the Directors, two years. Provided, however, that at the first election after the adoption of this By-Law, a President, one Vice-President, three Directors and a Treasurer shall be elected to serve for one year only, and one Vice-President and three Directors for two years; provided, also, that after the first election a President, one Vice-President, three Directors and a Treasurer shall be elected annually.

The term of each officer shall begin at the close of the Annual Convention at which such officer is elected, and shall continue for the period above named or until a successor is duly elected.

A vacancy in the office of President shall be filled by the senior Vice-President. A vacancy in the office of Vice-President shall be filled by the senior Director.

Seniority between persons holding similar offices shall be determined by priority of election to the office, and when these dates are the same, by priority of admission to membership; and when the latter dates are identical, the selection shall be made by lot. In case of the disability or neglect in the performance of his duty, of any officer of this Association, the Board of Direction shall have power to declare the office vacant. Vacancies in any office for the unexpired term shall be filled by the Board of Direction, except as provided above.

SEC. 4. The Board of Direction shall appoint the Secretary; it shall create such special committees as may be deemed desirable for the purpose of preparing recommended practice and standards concerning the proper use of cement for consideration by the

Association, and shall appoint a chairman for each committee. Four or more additional members on each special committee shall be appointed by the President, in consultation with the Chairman.

SEC. 5. It shall be the duty of the Board of Direction to audit the accounts of the Secretary and the Treasurer before each annual convention.

SEC. 6. The Board of Direction shall appoint a Committee on Nomination of Officers and a Committee on Resolutions, to be announced by the President at the first regular session of the annual convention.

SEC. 7. There shall be an Executive Committee of the Board of Direction, consisting of the President, the Secretary, the Treasurer and two of its members, appointed by the Board of Direction.

SEC. 8. The Executive Committee shall manage the affairs of the Association during the interim between the meetings of the Board of Direction.

SEC. 9. The President shall have general supervision of the affairs of the Association. He shall preside at the Annual Convention, at the meetings of the Board of Direction and the Executive Committee, and shall be ex-officio member of all committees.

The Vice-Presidents in order of seniority shall discharge the duties of the President in his absence.

SEC. 10. The Secretary shall perform such duties and furnish such bond as may be determined by the Board of Direction.

SEC. 11. The Treasurer shall be the custodian of the funds of the Association, shall disburse the same in the manner prescribed and shall furnish bond in such sum as the Board of Direction may determine.

SEC. 12. The Secretary shall receive such salary as may be fixed by the Board of Direction.

ARTICLE III.

MEETINGS.

SECTION 1. The Association shall meet annually. The time and place shall be fixed by the Board of Direction and notice of this action shall be mailed to all members at least thirty days previous to the date of the Convention.

SEC. 2. The Board of Direction shall meet immediately after the Convention at which it was elected, effect organization and transact such business as may be necessary.

SEC. 3. The Board of Direction shall meet at least twice each year. The time and place to be fixed by the Executive Committee.

SEC. 4. A majority of the members shall constitute a quorum for meetings of the Board of Direction and of the Executive Committee.

ARTICLE IV.

DUES.

SECTION 1. The fiscal year shall commence on the first of July and all dues shall be payable in advance.

SEC. 2. The annual dues of each member shall be five dollars ($5.00).

SEC. 3. Any person elected after six months of any fiscal year shall have expired, need pay only one-half of the amount of dues for that fiscal year; but he shall not be entitled to a copy of the Proceedings of that year.

SEC. 4. A member whose dues remain unpaid for a period of three months shall forfeit the privilege of membership and shall be officially notified to this effect by the Secretary, and if these dues are not paid within thirty days thereafter his name shall be stricken from the list of members. Members may be reinstated upon the payment of all indebtedness against them upon the books of the Association.

ARTICLE V.

RECOMMENDED PRACTICE AND SPECIFICATIONS.

SECTION 1. Proposed Recommended Practice and Specifications to be submitted to the Association must be mailed to the members at least thirty days prior to the Annual Convention, and as there amended and approved, passed to letter ballot, which shall be canvassed within sixty days thereafter, such Recommended Practice and Specifications shall be considered adopted unless at least ten per cent. of the total membership shall vote in the negative.

ARTICLE VI.

AMENDMENT

SECTION 1. Amendments to these By-Laws, signed by at least fifteen members, must be presented in writing to the Board of Direction ninety days before the Annual Convention and shall be printed in the notice of the Annual Convention. These amendments may be discussed and amended at the Annual Convention and passed to letter ballot by a two-thirds vote of those present. Two-thirds of the votes cast by letter ballot shall be necessary for their adoption.

SUMMARY OF THE PROCEEDINGS OF THE EIGHTH ANNUAL CONVENTION.

First Session, Monday, March 11, 1912, 8 p. m.

The Convention was called to order by the President, Richard L. Humphrey.

John Lyle Harrington, Past President, Engineers' Club of Kansas City, delivered an address of welcome on behalf of the Engineering Interests as follows:

In the dark ages of industry which extended well into the last century, it was the custom for every member of a craft, trade or profession to guard jealously and to keep closely secret every item of knowledge he or his associates had, in order to secure to himself the whole advantage of it. In addition to patenting an invention, it was common to keep secret every possible detail of the processes employed, and many businesses were based wholly upon secret formulæ which were closely held by members of a firm or family, often handed down from father to son, and the utmost precautions were taken to ensure that employees and even associates, as well as competitors, actual and potential, should be kept in ignorance of the methods employed or discoveries made. Even up to the present day it is dangerous in some establishments for an employee to ask too many questions regarding the methods of manufacture or materials he employs in his work, and in certain lines of manufacture a considerable remnant of this old secrecy remains. Here and there the possessor of a formula has, like the old alchemist, at the right moment dropped his fluid into a molten metal or added his mite to the production of important materials and kept the secret and profits thereof to himself.

But early in the last century the civil engineers of Great Britain met and formed an institute for the purpose of disseminating the knowledge acquired by its individual members; and in the latter half of the century the engineers of this country came to appreciate the advantages of co-operation. The first such organizations were few in number and comprehensive in scope, but gradually important groups working in special lines came to feel that the interests of the broad general organization were too varied to permit adequate consideration of the matters which specially occupied their attention and so they split off and organized societies of more limited scope. With the enormous development of industries based on the applied sciences special interests so increased in value and importance that the benefits to be derived from close association and active discussions of men engaged in them came to be generally understood, and group organizations grew apace.

In the course of time, manufacturers began to understand that by keeping to themselves knowledge of their specialities they encouraged like action on the part of their competitors—and each member of that branch of industry developed internally only. This limited the development of all and circumscribed the field of the industry. Gradually it became evident that the advancement of the industry resulting from the dissemination of knowledge of it among those engaged in it more than compensated for the advantage secured even by the most successful operation under the secret methods. As soon as this fact came to be fully realized the organization of special societies for special purposes multiplied rapidly. They reacted upon the industries and the accomplishments of one man so pointed out possibilities and so stimulated others that development has increased in geometric ratio.

Among these later organizations developed to special interests, the National Association of Cement Users has come to occupy an important place.

Cement in some form has been in use for many centuries, so many that its origin is lost in antiquity; but the development of Portland cement and the industries and types of construction dependent upon its use are of comparatively recent date. It is hard to believe that less than twenty years ago Congress gravely questioned the existence in this country of materials essential to the manufacture of first-class Portland cement, and debated whether, in view of that condition, it would be justifiable to impose so much duty on imported cement as would induce its manufacture in this country. Yet last year we produced nearly 78,000,000 barrels, which, at the existing low price prevailing, brought nearly $68,000,000 at the mills, and the value of constructions in which it was used probably reached nearly $500,000,000.

It is well, therefore, that the users of cement, the men who are responsible for construction of cement and concrete work worth one-half billion dollars per annum, should organize themselves into an association and meet to advance their special interests by the discussion of the work they are doing and the experiences they have gained. And while it is natural and right that interest and enthusiasm for their work should lead chiefly to the exposition of the successes achieved, it is quite as important that the defects found in the materials, in the constructions and in the methods employed should be exposed and discussed, for we learn as much—often more, if we are wise—from our failures as from our successes. And it is important that the limitations of materials and their uses be fully understood and bad results thus guarded against, and the character of the practices on the whole thus improved. The effort to meet the ever-pressing demand for cheap construction leads to many failures and to much consequent damage to the industry. To increase the stresses or reduce the quality of concrete in order to enable it to compare favorably with wood in first cost is an unwarranted, but far too common practice. If in any instance its many superior qualities do not justify the greater expenditure for good concrete safely stressed, then the cheaper materials should be employed. Wilful, foolhardy risks are responsible for some of the failures, but ignorance of all conditions governing concrete construction

is by far the larger factor, and it is the duty and purpose of this organization to expose bad construction as it is to urge good methods and good workmanship. Concrete construction is peculiarly liable to failure through both ignorance and carelessness. Through ignorance because its use in the simpler types of construction leads men to believe its employment universally simple; through carelessness because it is so amenable to the employment of unskilled labor which needs, but does not always receive, thorough and careful supervision. I well remember the county engineer who was confident he had made all the plans necessary for the construction of a reinforced concrete bridge when he had made a picture on which he had given general dimensions and shown the location of some reinforcement.

Failure to recognize the need of careful inspection of materials and supervision of workmanship is perhaps responsible for the larger portion of failures of concrete structures. Steel is manufactured by skilled men working under able superintendence and so continuously employed that they come to understand the processes thoroughly; whereas concrete is commonly manufactured by an itinerant, common laborer and often directed by a foreman whose only qualification is his ability to handle men. Constructions of wood, brick or stone may be fairly well inspected at any stage or after completion, but the character of concrete construction may not be determined until after the forms are removed, and remedying defects then disclosed is always difficult, often impossible. Too early removal of the forms, thus placing upon the concrete stresses which it is yet unfitted to bear; carelessness and unwise placing of reinforcement; inadequate provision for expansion and contraction due to changes of temperature; weak and leaky forms; inadequate tamping; loose methods of depositing concrete, both in air and under water; excessive dependence in designing upon empirical methods and fallacious load tests; these and many other difficulties must be guarded against and overcome. Much disappointment is certainly in store for the owners of staff and concrete buildings who have not fully appreciated the difficulties in securing construction which will resist the weather. There are many other difficulties which must be guarded against and overcome.

As the use of concrete becomes more general, more attention is given to the sightliness of structures built of it, but too often the efforts in this line are misdirected. It should be clearly recognized that this material is capable of excellent treatment peculiar to itself, and that it is an error to try to make it resemble other materials. The old Spanish structures of southwestern states and Mexico are beautiful because they are true. They were designed frankly to be built of concrete and have forms and finish suitable for that material. It may be that with the wider use and fuller development of concrete construction we may improve upon the work of these Spaniards, but we have not by any means equalled it as yet. The endeavor to meet the needs of the material has too frequently led to the adoption of extreme and grotesque forms and finishes which soon weary or offend.

We are gradually improving the finish of our concrete structures and recent work in this line is especially promising, but much remains to be done both for the appearance and for the weathering quality of concrete and staff

surfaces. Good finish adds to the cost, but the extra expense is surely justified.

The great advantages of concrete, durability, low cost as compared with other equally durable materials, ease of handling, ease of molding to the forms desired, resistance to fire, strength and homogeneity, adaptability to many uses, both with and without reinforcement, resistance to acids and decay, attractiveness of appearance and the wide distribution of its constituent materials, improvements in methods of construction, and the steadily increased scarcity and cost of good timber ensure continual increase in the use of concrete. But perhaps the greatest argument in favor of fireproof concrete buildings is the enormous fire loss this country sustains through the excessive use of wood and the equally great cost of fire insurance. Our losses by fire have become a national disgrace.

The influence of this Association should be very great in combating the tendency of some constructors to secure business and profit by reducing the quality of work. This course surely militates against the interest of cement users as a whole and it brings their work as a whole into disrepute. This was clearly exemplified recently at Los Angeles where the city engineer recommended creosoted instead of reinforced concrete piles for dock construction and was able to justify his position by citing about a dozen failures of docks which were supported on reinforced concrete piles. And generally speaking, any improper use of the material of construction affects adversely the interests of all who engage in the use of that material. There is, therefore, a large opportunity for this Association to benefit greatly the interests of its members and to benefit the country at large by compelling a high standard of ability and integrity among cement users.

It is thus apparent that this Association has met to deal with subjects of the largest consequence and the effects of its discussions will be to improve throughout this country and other countries the practices of concrete design and construction. The work of this Association is, in no inconsiderable measure, the work of the Engineer. He shares largely in the work and the responsibilities and the benefits of this convention; hence it is with pleasure and cordial good will that on behalf of the Engineers of Kansas City, I extend a hearty welcome to the National Association of Cement Users.

An address of welcome on behalf of the Concrete Interests was made by F. W. Fratt, President of the Union Bridge and Terminal Railroad Company.

The President responded:

The Convention has this year been fortunate in having two eminent engineers express thoughts that will be of good service to the Association in its future work. I am sure that you all join me in reciprocating the good wishes which Mr. Harrington and Mr. Pratt have extended to us, and in the hope that our deliberations may prove of interest and value.

The following committees of the Convention, appointed by the Executive Board, were announced by the President:

Committee on Nomination of Officers:

E. J. Moore, *Chairman*, New York, N. Y.
D. A. Abrams, Ames, Iowa.
John L. Conzelman, St. Louis, Mo.
B. F. Lippold, Chicago, Ill.
L. T. Sunderland, Kansas City, Mo.

Committee on Resolutions:

Rudolph P. Miller, *Chairman*, New York, N. Y.
P. H. Bates, Pittsburgh, Pa.
Allen Brett, New York, N. Y.
F. L. Williamson, Kansas City, Mo.
Percy H. Wilson, Philadelphia, Pa.

A paper on "The Use of Reinforced Concrete in Hypochlorite Water Purification Works" was read by Walter M. Cross.

George E. Tebbetts presented a paper on "The Use of Concrete in the New Union Station at Kansas City, Mo.," which was followed by a discussion.

The meeting adjourned until Tuesday at 10.30 A. M.

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TUESDAY, MARCH 12, 1912, 10.00 A. M.

Meeting of the Sections on Measuring Concrete, Nomenclature, and Specifications and Methods of Tests for Concrete Materials.

President Richard L. Humphrey in the chair.

The meeting took the form of a discussion on the deposition of mortar with compressed air, its effectiveness and various methods of application.

SECOND SESSION, TUESDAY, MARCH 12, 1912, 10.30 A. M.

President Richard L. Humphrey in the chair.

The report of the Committee on Specifications and Methods of Tests for Concrete Materials was, in the absence of the Chairman, Sanford E. Thompson, presented by Cloyd M. Chapman.

Wm. M. Kinney read a paper on "Aggregates for Concrete,"

which was followed by a Topical Discussion on Concrete Aggregates.

In the absence in Europe of the Chairman, Robert A. Cummings, Edw. D. Boyer presented the report of the Committee on Measuring Concrete, which was followed by a discussion. The Proposed Standard Method of Measuring Concrete was referred back to the Committee, with instructions for a revision in the light of the discussion and report at the next annual Convention.

THIRD SESSION—TUESDAY, MARCH 12, 1912, 8.00 P. M.

President Richard L. Humphrey in the chair.

The annual address of the President, entitled "The Use of Concrete in Europe," was delivered by Richard L. Humphrey.

The following papers were then read and discussed:

"The Design and Construction of the Hollow Reinforced Concrete Dam of the Portland Railway Light and Power Company," by Herman V. Schreiber.

"Cement Coatings in Color," by F. J. Morse.

The report of the Committee on Concrete Surfaces was presented by the Chairman, L. C. Wason. On motion the proposed Standard Method for Tests of Waterproofing was referred to the Committee on Specifications and Methods of Tests for Concrete Materials, for report. Consideration of the proposed Standard Specification for Portland Cement Stucco was deferred until a later session. The changes in the last general report of the Committee were approved.

The report of the Committee on Insurance was, in the absence of the Chairman, Wm. H. Ham, read by title.

The meeting then adjourned until Wednesday at 9.30 A. M.

FOURTH SESSION—WEDNESDAY, MARCH 13, 1912, 9.30 A. M.

President Richard L. Humphrey in the chair.

In the absence of the author, Sanford E. Thompson, the President read the paper entitled "The Practical Design of Reinforced Concrete Flat Slabs."

The report of the Committee on Reinforced Concrete and Building Laws was presented by the Chairman, Alfred E. Lindau, and Arthur N. Talbot, and was followed by a discussion.

The President then presented the annual report of the Executive Board, submitting a proposed revision of the By-Laws, which were amended and ordered to letter ballot. The report was approved.

The change of name of the Association was discussed and on motion the matter was referred to the Executive Board, with authority to effect a change in name if deemed expedient for the best interests of the Association.

The Committee on Nomination of Officers presented the following nominations, which were unanimously approved and the Secretary instructed to cast the ballot for their election:

President, Richard L. Humphrey, Philadelphia, Pa.
First Vice-President, Edward D. Boyer, Catasauqua, Pa.
Second Vice-President, Arthur N. Talbot, Urbana, Ill.
Third Vice-President, Edward S. Larned, Boston, Mass.
Fourth Vice-President, Ira H. Woolson, New York, N. Y.
Treasurer, Henry C. Turner, New York, N. Y.

The time and place of the Ninth Annual Convention was referred to the Executive Board with power to act.

The meeting then adjourned until 3 P. M.

FIFTH SESSION—WEDNESDAY, MARCH 13, 1912, 3.00 P. M.

President Richard L. Humphrey in the chair.

The following papers were read and discussed:

"The Testing of Reinforced Concrete Buildings under Load," by W. A. Slater.

"The Design of Concrete Flat Slabs," by Frank J. Trelease; in the absence of the author read by Alfred E. Lindau.

"The Present Status of Unit Construction," by James L. Darnell.

In the absence of the author, Theodore H. Skinner, the paper on "A Fireproof School of Concrete," was read by title.

The meeting then adjourned until 8.00 P. M.

SIXTH SESSION—WEDNESDAY, MARCH 13, 1912, 8.00 P. M.

President Richard L. Humphrey in the chair.

The President introduced the Honorable Darius A. Brown, Mayor, who extended an address of welcome to Kansas City as follows:

Mayor Brown.—I am glad of the opportunity to appear before you and express my appreciation of the fact that men should gather together in this city for the purpose of discussing the matters which have brought you here. We have heard about the Stone Age, the Wooden Age and the Iron Age, and I think it is the consensus of opinion that we are now in the Concrete Age and that all of the improved structures are coming to be made of this material. We are also in the age of progress and advancement; we are in an age when people have adopted the idea that if a thing is worth doing at all it is worth doing well and that is the reason why in every branch of human industry, in business, in commerce, in the sciences and in the professions, they gather together periodically for the purpose of discussing the ways and means of better doing the business in which they are engaged.

I am satisfied that the result of your deliberations will not only be of benefit to you in your particular business but will be of benefit to the community in which you live and benefit to the American people as a whole. I hope one of the results of your deliberations will be that the use of concrete will become so perfected that it will not only give us more comfortable buildings and better and more ornamental structures but will decrease the cost of those materials to the people; because that is one of the great problems to be solved.

On behalf of the people of Kansas City I want to extend to you a very cordial welcome and trust that you will not only be benefited by your deliberatons but that you will have some pleasure while staying in our city. I thank you.

The President responded:

I am sure we all appreciate the welcome that has been extended by His Honor. A city of this size in whose immediate vicinity there is a production of Portland cement of about one-tenth of that of the entire country, is a good place in which to hold deliberations of this character. In accepting the hospitality of this city we do so with a feeling that we will be benefited. Our conventions have been held heretofore east of the Mississippi River, and I believe that the mingling of the eastern and western ends of this great country cannot help but be beneficial to all. I know you will join me in extending to Mayor Brown hearty thanks for his welcome.

A paper on "The Use of Reinforced Concrete on the Wabash Railroad," was presented by A. O. Cunningham,

The report of the Committee on Treatment of Concrete Surfaces was taken up and on motion the proposed Standard Specification for Portland Cement Stucco was received as information and the Committee instructed to confer with other Associations, in order to reach an agreement as to a specification to be presented at the next Convention.

The following papers were then read and discussed:

"The Design and Construction of a Reinforced Concrete Dome, 220 Foot Span," by S. J. Trauer; in the absence of the author read by the President.

"The Design of Concrete Grain Elevators," by E. Lee Heidenreich.

"The Suitability of Concrete for Gas Holder Tanks," by Herbert W. Alrich; in the absence of the author presented by the President.

"The Necessity of Field Tests for Concrete," by Fritz E. Von Emperger; in the absence of the author presented by the Secretary.

The meeting adjourned until Thursday at 10.30 A. M.

THURSDAY, MARCH 14, 1912, 10.00 A. M.

Meeting of the Section on Treatment of Concrete Surfaces.

President Richard L. Humphrey in the chair.

The meeting took the form of a topical discussion on the coloring of concrete surfaces, contraction, dusting of floors, etc.

SEVENTH SESSION—THURSDAY, MARCH 14, 1912, 10.30 A. M.

President Richard L. Humphrey in the chair.

The following papers were read and discussed:

"Concrete Highway Bridges," by William Scott Gearhart.

"Concrete Bridges," by Daniel B. Luten.

"Flat Slab Bridges," by William H. Finley; in the absence of the author read by the President.

The meeting adjourned until 8.00 P. M.

EIGHTH SESSION—THURSDAY, MARCH 14, 1912, 8.00 P. M.

President Richard L. Humphrey in the chair.

The paper on "The Necessity for Good Roads," by Logan Waller Page, was in the absence of the author presented by E. L. Eldredge.

The following papers were then presented:

"The Necessity of National Aid in Good Roads," by H. C. Gilbert.

"Cement Paving as Constructed at Mason City, Iowa," by F. P. Wilson.

"An Improved Concrete Pavement," by E. W. Groves.

The report of the Committee on Roadways, Sidewalks and Floors, was presented by the Chairman, C. W. Boynton, and the following action taken:

Proposed revisions of the Standard Specifications for Concrete Road and Street Pavements ordered to letter ballot.

The proposed revision of the Specifications on Sidewalks, Curb and Gutter, and the proposed new Specifications for Plaiin and Reinforced Concrete Floors, were considered and referred to the Committee for report at a later session.

The meeting then adjourned until Friday at 10.30 A. M.

FRIDAY, MARCH 15, 1912, 9.00 A. M.

Meeting of the Section on Roadways, Sidewalks and Floors; C. W. Boynton, Chairman of the Section, in the chair.

The meeting took the form of a discussion on concrete roads, concrete floors, the prevention of dusting of floors, concreting in freezing weather, etc.

NINTH SESSION—FRIDAY, MARCH 15, 1912, 10.30 A. M.

President Richard L. Humphrey in the chair.

The following papers were read and discussed:

"Concrete Fence Posts," by L. J. Hotchkiss.

"The Design of Reinforced Concrete Retaining Walls," by John M. Meade.

"Advantages and Durability of Cement Sewer Pipe," by Gustave Kaufman; in the absence of the author presented by the President.

"Methods of Testing Cement Pipe," by Duff A. Abrams; in the absence of the author presented by W. A. Slater.

"The Manufacture and Use of Cement Drain Tile," by Charles E. Sims.

The report of the Committee on Cement Products and Building Blocks was in the absence of the Chairman, Percy S. Hudson, presented by Clarence K. Arp. On motion the Proposed Standard Recommended Practice for Cement Tile was referred to letter ballot.

The Committee on Roadways, Sidewalks and Floors, C. W. Boynton, Chairman, reported back the matters referred to it and on motion the following were referred to letter ballot:

Proposed Revisions of the Standard Specification for Portland Cement Sidewalks.

Proposed Revisions of the Standard Specifications for Portland Cement Curb and Curb and Gutter.

Proposed Standard Specifications for Plain Concrete Floors.

Proposed Standard Specifications for Reinforced Concrete Floors.

The report of the Committee on Nomenclature was in the absence of the Chairman, Peter Gillepsie, presented by Frank C. Wight and was on motion accepted as information.

The Committee on Education reported progress.

The Convention then adjourned until Saturday at 10.30 A. M.

SATURDAY, MARCH 16, 1912, 9.30 A. M.

Meeting of the Section on Building Blocks and Cement Products, President Richard L. Humphrey in the chair.

The meeting discussed the materials, methods of manufacture and tests of cement drain tile.

TENTH SESSION—SATURDAY, MARCH 16, 1912, 10.30 A. M.

President Richard L. Humphrey in the chair.

The paper by George Gibbs on "Some Notes on the Value and Comparative Cost of Reinforced Concrete Telegraph Poles," was in the absence of the author presented by the President.

The President read in the absence of the author, Robert A. Cummings, the paper on "The Making and Driving of Reinforced Concrete Piles Within Six Days."

The paper by W. J. Towne on "Concrete Fence Posts," was in the absence of the author read by title.

A paper on "Comparative Tests of the Strength of Concrete in the Laboratory and in the Field," was presented by R. J. Wig.

The following papers were, in the absence of the authors, read by title:

"Field Inspection and Tests of Materials for Reinforced Concrete," by G. H. Bayles.

"Unit Cost of Reinforced Concrete for Industrial Buildings," by C. S. Allen.

"Notes on the Deformation in the Webs of Rectangular Concrete Beams," by H. C. Berry.

The President then presented a paper by Alfred D. Flinn on "The Use of Cement for Protecting Steel Pipes Along the New York Aqueduct."

Robert F. Havlik presented a paper on "Modern Methods of Manufacturing Concrete Products," which was followed by a discussion.

The meeting then adjourned until 8.00 P. M.

ELEVENTH SESSION—SATURDAY, MARCH 16, 1912, 8.00 P. M.

President Richard L. Humphrey in the chair.

The paper on "The Use of Concrete in the Fourth Avenue Subway, Brooklyn, N. Y.," was in the absence of the author, Frederick C. Noble, presented by the President.

W. A. Collings presented a paper on "Reinforced Concrete in Agriculture."

The paper by S. B. Williamson on "The Handling of Concrete in the Construction of the Panama Canal," was in the absence of the author, presented by the President.

The following papers were then read and discussed:

"The Present Status of Iron Portland Cements," by P. H. Bates.

"Iron Portland Cement," by Herman E. Brown; in the absence of the author, presented by the President.

The Committee on Resolutions, Rudolph P. Miller, Chairman, presented the following resolutions, which were unanimously adopted:

(a) *Resolved*, That a committee be appointed by the Executive Board to consider the form of all standard specifications or recommended practice issued by this Association with a view to securing uniformity so far as practicable.

(b) *Resolved*, That a committee of five members of this Association, of which one member shall be Chairman of the Committee on Specifications and Methods of Tests for Concrete Materials, be appointed by the President to plan a comprehensive and systematic investigation of the aggregates used for concrete and to interest State Universities, Experiment Stations and other laboratories in carrying out the same.

(c) *Resolved*, That the Executive Board be instructed to consider the advisability of appointing a Committee to report on Standard Specifications for Concrete Highway Bridges and Culverts.

(d) *Resolved*, That the Committee on Nomenclature be instructed and empowered to extend its work to include the standardization of the size of drawings, the symbols used on same and the graphical representation of details.

Resolved, That the Committee on Cement Products be instructed to consider the suggestions and criticisms on building block specifications offered at this Convention, to confer with the Committee on Reinforced Concrete and Building Laws with a view to reconciling there commendations of the two committees, and to report revised specifications to the next convention.

(e) *Resolved*, That a report be submitted to the next Convention on Standard Specifications for Concrete Fence Posts and that the Executive Board consider the advisability of having this done by a sub-committee of the Committee on Cement Products or by a separate committee.

(f) *Resolved*, That the thanks of this Association are hereby tendered the officials and the representatives of the local engineering and concrete interests for their hearty welcome, to the citizens of Kansas City for their co-operation in making this, the Annual Convention, a notable success, and to the guests of the Association for their assistance in this success by the contribution of their interesting and valuable papers.

(*g*) *Resolved*, That the thanks of this Association are hereby tendered to the members who have aided by the presentation of papers, to the several committees whose efforts have added this meeting to the long series of successful conventions, to the local and technical press whose recognition of the work of this organization is gratefully acknowledged, and to its officers but particularly to its President, Mr. Richard L. Humphrey, for his untiring devotion to the interest and welfare of this Association.

The President thereupon declared the meeting adjourned, *sine die*.

NATIONAL ASSOCIATION OF CEMENT USERS.

PROCEEDINGS

OF THE

EIGHTH CONVENTION

This Association is not responsible, as a body, for the statements and opinions advanced in its publications.

EUROPEAN PRACTICE IN CONCRETE CONSTRUCTION.

ANNUAL ADDRESS BY THE PRESIDENT,
RICHARD L. HUMPHREY.*

Two years ago, in speaking on the use of concrete in Europe, I discussed in a general way the progress that was being made and contrasted the conditions prevailing in various parts of Europe. It was also pointed out that in the artistic treatment of concrete the foreign engineer and architect undoubtedly showed greater skill than was shown in this country, and therefore obtained much more pleasing results. I further commented on the fact that the development of the use of concrete in certain countries was very much handicapped by restrictive building laws. It was my good fortune to again visit Europe last year and to inspect extensively various structures of concrete, covering the most important work west of the Russian boundary. I shall use this opportunity for enlarging upon my former address, pointing out the development and essential points of difference at the present time in reinforced concrete construction in this country and in Europe.

This visit to Europe and the inspection of concrete construction during the closing months of last year, was under more favorable conditions than on the occasion of my previous trip, in that I was a guest generally of the concrete associations, whose officers did all in their power to show me everything of interest. This was particularly true of my visit to Austria, where, as the Presi-

* Consulting Engineer, Philadelphia, Pa.

dent of this and Honorary Member of the Austrian Concrete Association, I was the guest of the latter during my entire stay in that country, and was under the devoted personal guidance of its Director, Karl Bitner. This gentleman, as you know, was a delegate to our New York Convention and one of the speakers at the banquet, and I wish again to express my heartiest thanks and appreciation for the courtesies extended to me and the unusual efforts which he made to render my visit a pleasant and profitable one. It is certainly true that this opportunity of inspecting the concrete buildings in Austria was a valuable one for the reason that some of the best examples of concrete construction are to be found in that country.

The Austrian Concrete Association has always manifested a great interest in the work of our Institute and showed a willingness to co-operate with us in every possible way. The unusual character of the program laid out and the rapidity with which various pieces of work were inspected was, I think, intended as a tribute to our American characteristics. I submit this interesting program. (See opposite page.)

A feature that impressed me most favorably was that my visits to the various buildings had been arranged for in advance and upon scheduled time. On our arrival at the building we were met by the architect or his representative, the builder, and the engineer in charge; in many instances the plans of the structure were tacked up at some convenient point, and before the building was inspected a representative who spoke English explained the particular points of interest in the structure. On Thursday night I was the guest of the Austrian Concrete Institute, the Austrian Association of Cement Manufacturers, Austrian Society of Engineers and Architects and the Austrian Clay Products Association. My here recorded acknowledgment of the signal honor conferred upon me but inadequately expresses the depth of my gratitude and the extent of my appreciation. The precision with which the progran was carried out, the completeness of the details and the warm hospitality extended to me by all those I met, has left its permanent record—one that I shall never forget.

During my visit to England it was my privilege to address the Concrete Institute in London, on October 26, 1911, on the subject of "Fireproofing" for which I was honored by the award of the Institute medal.

ÖSTERREICHISCHER BETONVEREIN

WIEN, IV/2, MÖLLWALDPLATZ 4

TELEPHONE Nr. 10.597 TELEPHONE Nr. 10.597

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ZEITEINTEILUNG

ANLÄSSLICH

DES BESUCHES DES PRÄSIDENTEN DER NATIONAL ASSOCIATION OF CEMENT USERS, HERRN RICHARD L. HUMPHREY, CONSULTING ENGINEER, PHILADELPHIA.

Donnerstag, 21. September, 1911.

9	Uhr	30	Min.	Abfahrt vom Hotel Bristol:
9	"	35	"	Kärntnerhofbasar, I. Kärntnerstrasse (Ed. Ast & Cie.);
10	"	20	"	Wohn- und Geschäftshaus, I. Weihburggasse 7 (Plachy & Co.);
10	"	30	"	Wohn -und Geschäftshaus, I. Weihburggasse 9 (G. A. Wayss & Cie.);
10	"	40	"	Wohn- und Geschäftshaus, I. Weihburggasse 10 (N. Rella & Neffe);
10	"	50	'	Lazzenhof, I. Rotenturmstrasse, verlängeter Fleischmarkt (Kontrollbalkenversuch) (k. k. Oberbaurat Dr. Ing. Fritz von Emperger — Chefingenieur Richard Wucżkowski)
11	"	30	"	Wiener Urania, I. Aspernplatz (G. A. Wayss & Cie.);
11	"	55	"	Dreilaufferhaus, I. Kohlmarkt, Ecke Herrengasse (Pittel & Brausewetter);
12	"	00	"	Wiener Bankverein, I. Schottenring (Ed. Ast & Cie. und N. Rella & Neffe);
12	"	30	"	Lunch im Rathauskeller;

2 Uhr 00 Min.	Autogarage, II. Rembrandtstrasse 29 (Janesch und Schnell);
2 " 40 "	Basaltoidpflaster (2 Jahre alt), XVI. Hasnerstrasse—Richard Wagner-Platz (Basaltwerke Radebeule);
2 " 55 "	Kirchenbau, XVI. Herbststrasse 65 (Max Emer & Cie.);
3 " 25 "	Zentralpalast, VI. Mariahilferstrasse — Ecke Kaiserstraffe (G. A. Wayss & Cie.);
4 " 00 "	Gewerbliche Fortbildungsschule, VI. Mollardgasse (N. Rella & Neffe);
4 " 20 "	Schokoladefabrik Stollwerck, V. Gaudenzdorfer Gürtel 43, 45 (Pittel & Brausewetter);
4 " 30 "	Schule, V. Margaretenstrasse 103 (N. Rella & Neffe);
5 " 15 "	Kronenbrotwerke, X. Siccardsburggasse 83 (Ed. Ast & Cie.);
	Während der Fahrt zu besichtigen:
	Kabelblocklegung für die Telephonleitungen, Unterleitung der Strassenbahn, Stadtbahn als Untergrund- und Hochbahn etc.;
7 " 00 "	Besuch des k. k. Hof-Operntheaters;
	Souper.

Freitag, 22. September.

8 Uhr 00 Min.	Fahrt nach Berndorf, Besichtigung der Kirche, Weiterfahrt nach Weissenbach, Besichtigung der Kunststeinfabrik; Adolf Baron Pittel
11 " 30 "	Lunch;
1 " 45 "	Weiterfahrt;
3 " 00 "	Besichtigung der Zementfabrik Achau;
4 " 30 "	Besichtigung der Betriebsanlagen der Wienerberger Ziegelfabriks- und Baugesellschaft (Keramitfabrik und Seidlbalkenerzeugung), X. Triesterstrasse 100;
8 " 00 "	Souper am Cobenzl, angeboten vom Österreichischen Betonverein, Verein Österreichischer Zementfabrikanten, Österreichischen Ingenieur- und Architektenverein und Österreichischen Tonindustrieverein (bei schlechtem Wetter Souper im Künstlerzimmer des Restaurants Hopfner).

Samstag, 23. September.

8 Uhr 00 Min.	Abfahrt vom Hotel Bristol:
8 " 30 "	Versuchsplatz des Eisenbetonausschusses des Österreichischen Ingenieur- und Architektenvereines, Wien, XIX. Muthgasse;
9 " 15 "	Schleusenanlagen in Nussdorf;
9 " 30 "	Wasserturm am Bahnhof Heiligenstadt (N. Rella & Neffe);
9 " 45 "	Zigarettenpapierfabrik Schnabl, XIX. Heiligenstadt (N. Rella & Neffe);
10 " 05 "	Brücke im Zuge der Rampengasse, XIX. (H. Rella & Co.);
10 " 20 "	Wagenhalle der städtischen Strassenbahnen, XVIII. Währinger Gürtel (N. Rella & Neffe);
10 " 45 "	Physikalisches Institut, IX. Währingerstrasse — Waisenhausgasse (Ed. Ast & Cie.);
11 " 30 "	Sargfabrik, XIII. Matznergasse 8 (A. Porr);
11 " 50 "	Technisches Museum für Industrie und Gewerbe, XIV. Ecke Winckelmannstrasse und Linzerstrasse (A. Porr);
12 " 30 "	Basaltoidflaster, XIII. Schönbrunner Schlossstrasse (Basaltwerke Radebeule);
12 " 45 "	Lunch im Parkhotel (Hietzing);
2 " 15 "	Landes-Heil- und Pflegeanstalt Steinhof;
3 " 30 "	Kohlenturm und Koksseperationsanlage im Gaswerke XXI. Leopoldau (H. Rella & Co.);
4 " 30 "	Siloanlage nebst Weichraum und Lagerraum in der Malzfabrik Hauser & Sobotka, XXI. Stadlau (H. Rella & Co.);
	Sehenswürdigkeiten von Wien;
8 " 00 "	Souper Venedig in Wien.

Eventuell:

Sonntag, 24. September.

Unter Führung des Herrn Ingenieur E. A. Westermann, Chef der Firma Wayss, Westermann & Cie., Graz:

Fahrt über den Semmering nach Graz, Besichtigung des Landeskrankenhauses Graz und der Brückenobjekte an der Bahnstrecke Weiz-Birkfeld.

Montag, 25. September.

Fahrt nach Retznei, Besichtigung der Zementfabrik, Weiterfahrt nach Adelsberg, Besichtigung der Adelsberger Grotte;

Fahrt nach Triest, Besichtigung der Hangarbauten, Fundierungen etc.

Dienstag, 26. September.

8 Uhr 00 Min.	Excelsior Palace Hotel (Wayss' Westermann & Cie.);
8 " 30 "	Greinitz (Masorana & Comel);
8 " 45 "	Riunione Adriatica (Ast. & Cie.);
9 " 00 "	Hanger -Bauten (Wayss' Freitag & Meinong);
10-12 "	Rundfahrt im Hafen;
Eventuell:	Souper in Opcina;
	Fahrt nach Miramare.

The various countries of Europe are progressing in the use of concrete, but many of the large cities are still handicapped by restrictive building laws; particularly is this true of London, where only recently have the London County Council Regulations permitted the erection of structures that might be termed reinforced concrete. Most continental countries show far greater skill in the application of concrete than is shown in England, where the British conservatism has resulted in heavy structures of very simple application. This is also true for the most part of the various cities in Germany. In France, in Belgium, and particularly in Austria a wider and less conservative grasp in the use of this material has resulted in the erection of structures

FIG. 1.—THE CONCRETE INSTITUTE MEDAL.

which are not equaled anywhere in the world. Certainly at the present time in Vienna I believe one may find the most extensive use of concrete that is to be found either in this country or Europe. The city of Hamburg is perhaps second only to Vienna in the number of its reinforced concrete buildings, and these two cities are the most progressive spots in Europe. I observed on this trip that more concrete buildings were in evidence in the outskirts of London and other English and continental cities than on my previous visit; in this country the same is true, probably for the same reason, viz., that the laws governing the erection of buildings are more liberal outside of than inside of the larger cities.

The development of the use of concrete is certainly much greater in Europe than it was two years ago. I do not think, however, that Europe as a whole shows as great a development

as is to be found in this country. In certain parts of Europe, Austria, for example, is shown a greater knowledge and skill in the use of this material. In England, especially in London, where there were practically no concrete buildings to be found on my visit two years ago, there are now to be found many structures of reinforced concrete; although it is true that these are not within the limits of the authority of the London County Council, which permits only reinforced concrete for floors with masonry-bearing walls, and to a limited extent for columns. It is probable, since a recent revision of the London County Building regulations permits the use of reinforced concrete, that from now on many structures will be erected of reinforced concrete.

The materials in Europe available for use in concrete are still relatively more expensive than labor. As a result, the designer, for purposes of economy, finds it desirable to so shape his forms as to eliminate as much as possible all material which is not required for structural or protective purposes. The effect of this is to render the structure less massive and more pleasing in appearance. The abundance of extremely cheap unskilled labor and the presence of low-priced technically-trained labor is one of the great advantages in the erection of concrete structures in Europe. This is particularly true as to the foremen and labor bosses who, in many cases, especially in Germany, are technically-trained men, which is of course unusual in this country.

Another feature which tends to increased efficiency in the erection of concrete structures is the fact that there are governmental regulations which apply with sufficient rigidity to territory outside of the large cities. In large cities the regulations are necessarily much more rigid. There is a wholesome respect for the law throughout Europe—which is lacking in this country—with the result that each person concerned conscientiously endeavors to erect the structure in full accordance with the building regulations. In some countries, especially Germany, a contractor who is found guilty of dishonest practices loses caste and becomes discredited, which is, after all, the most effective way of preventing the construction of dishonest structures. I recall particularly a case in Stuttgart where what might properly be called a "Quantity" engineer, having assumed responsibility for the erection of the building, was arrested and sentenced to several years imprisonment by reason of the collapse of the structure of

which he had assumed the responsibility. Another effective way of dealing with this subject is practiced in France, especially in Paris, where the building department acts merely as a custodian of plans; the law placing the responsibility for the structure on the architect, contractor and owner. Under the law, these three parties are held to be responsible for the collapse of the structure until their innocence is established. The use of cheap labor, especially women, to carry mortar and concrete in small tubs on the head, seems to be the usual method but hardly an effective one for handling concrete. In the erection of but two buildings did I see used the elevating machine, so commonly used in this country, for handling concrete.

The presence of a large number of women laborers on concrete structures was always a source of interest to me as was also the pittance that these women were paid for a day's work. A recent lecturer in New York stated that in African hunting expeditions the camp followers received about sixty cents a month. While these women do not receive so little, yet when you consider that in the one case the men were furnished their food, while in the other, the women had to supply it themselves, the wage paid the women (in some countries equal to 16 cents and as low as 12½ cents a day) is extraordinarily low and you can appreciate why elevating and conveying machinery is doubtless more expensive than labor. The almost universal limit of about 5 stories or 22 metres in the height of all structures renders elevating and conveying machinery relatively unimportant. When, however, speed in the erection of concrete structures becomes important, Europe will be obliged to resort to mechanical means for elevating and conveying concrete, as the method of carrying concrete in tubs on the heads of the laborers is too slow for proper continuous placing.

Fig. 2 is a view of the memorial church at Berndorf, the industrial village of the Krupp works, just outside of Vienna, in which will be seen a number of the women laborers who are engaged in carrying mortar and concrete in the manner above described. These women, in spite of their skirts, are able to climb ladders with almost the same speed as men.

The use of round timbers instead of sawed, as studs for forms and scaffolding, is quite general. Where a splice is necessary, the two parts are tied together with rope or chain. They do

not use nails, therefore, these timbers can be used over and over again and last for such a length of time as to make the cost relatively very low. Another interesting construction detail is the method of splicing uprights through the medium of two iron rectangular iron bands surrounding the uprights, which are clamped tightly together by wedges between the inside of the band and the face of one of the uprights, which is a very simple and readily adjustable device. This particular device was used on the props in the construction of the Commercial Museum in Vienna.

The development in the use of reinforced concrete telephone

FIG. 2.—DOME MEMORIAL CHURCH IN BERNDORF, AUSTRIA.

and telegraph poles is even greater than it was on my previous visit and the experience gained has brought forth many ornamental and efficient poles; the tendency for purposes of economy is towards a hollow pole, this being of greater necessity in Europe than in America; and in my judgment the cost of the concrete pole must be materially reduced, to effectually compete with the wooden pole in this country. The view shown (Fig. 3) of the centrifically molded circular poles in Bad Kösen indicates the uniformity and symmetrical shape of this pole. This circular form, however, is not necessary, and there are many hollow poles of ornamental character of square or octagonal design, notably

four in Dresden, in the Exposition of Hygiene. The ornamental poles on the Augusta Bridge in Dresden further exemplify the beauty of this type of pole, the hollow interior affording an ideal place for running the electrical wires.

The method on making the hollow reinforced concrete pole

FIG. 3.—CENTRIFUGALLY MOLDED CIRCULAR TOWN ELECTRIC LIGHT POLES, IN BAD KÖSEN, GERMANY.

consists in placing the mold filled with concrete in a machine rotated at the rate of 600 revolutions per minute. The effect of this high speed is to force the concrete against the walls of the mold by centrificial action, gradually compacting the concrete and forming a hollow space in the center of the pole in which

the excess water and laitance gathers producing an exterior surface of a hard and uniform texture which greatly adds to the appearance of the pole. The molds are generally kept on the pole until properly hardened.

The poles erected in connection with the Danish railways in Copenhagen, Denmark, are hollow, 36 ft. high, and were molded by the hand process and the tests showed them to be very stiff and capable of resisting high loads.

It has been found in Europe, especially where lumber is becoming scarce, that reinforced concrete poles are much more efficient and less costly than wooden poles; that the maintenance of the line is less expensive and the permanent life of the pole much greater. It appears to me to be evident that in order to effect the desired economy in the cost of manufacture of this class of concrete products, it is necessary to turn out a great many each day; inasmuch as they are coming into general use in Europe, there is a constant decrease in the cost of manufacture.

The poles illustrated in Fig. 4 are those at the plant of R. Wolle in Leipsic, Germany. There seems to be a general use for concrete poles for carrying high-tension lines, especially where the pole must be of considerable height. It is claimed that the cost of maintenance of such lines is very much less than for wooden poles. This probably accounts for their popularity.

The continued use of reinforced concrete poles in this country leads to the belief that as the number of poles and skill in making them increases they will become more serious competitors of the wooden pole and will in time replace the other forms of telephone, telegraph and electric transmission poles. They can be molded to suit the particular conditions of almost any height and can be so anchored in the ground as to enable them to maintain a rigid position in almost all soils. The high tension transmission line poles of reinforced concrete, used in connection with the Pennsylvania Railroad tunnels,* which were erected on a mattress in the marshy land of the approaches on the New Jersey side, are an illustration of the superior excellence of this type of pole.

Another matter which was of considerable interest to me was the form of chimney developed by Captain F. Möhl in Copenhagen. This consists of a four-leaf-clover section at the base

* See *Proc. N.A.C.U.*, Vol. VIII, p. 759.

which gradually merges into a circular at the top. A number of these chimneys have been erected, some of considerable height, and it is said that they are more economical and much more suitable than the ordinary circular stack.

The reinforced concrete barges shown in the illustration were photographed (Fig. 5) at the place of manufacture in Livorno, Italy. These particular barges were used for handling coal and seemed to be in every way thoroughly satisfactory. When the Italian government first used reinforced concrete for armor plate,

FIG. 4.—TYPES OF REINFORCED CONCRETE POLES AT PLANT OF R. WOLLE, LEIPSIC, GERMANY.

there was much amusement manifested in this country and the average person believed that the weight of this material would sink the boat. Its use, therefore, to form the entire hull of a boat would seem even more quixotic. Barges and other vessels, especially battleships, are made entirely of steel, which is heavier than concrete; and when you consider that the floating of the vessel is a question of buoyancy depending on the lightness of the material and character of the air-tight compartments, it is evident, I think, that any material properly designed will be

sufficiently buoyant for practical purposes. In the case of the armament of concrete, which is much lighter than steel, the thickness can be much greater and the toughness of the former material renders it a better protection. I believe with the development of the art of reinforced concrete boat construction, these vessels will in the future come into general use and will prove most serviceable and economical, both as to first cost, maintenance and durability.

It is not so very many years ago that the concrete barge was a novelty and regarded by many as a freak application of cement. However, a few years' trial of these boats has resulted in striking economy and in many places I found concrete barges being used,

FIG. 5.—REINFORCED CONCRETE BARGE FACTORY AT LIVORNO, ITALY.

especially of the canal boat type, for handling coal and other materials. It has been found that the durability and serviceability of these boats render their ultimate cost very much less than boats made of any other material. There have been a number of such boats used in this country, notably in connection with the construction of the Panama Canal, and in my judgment there will be an even greater use of them in the future.

The constant study of the reinforced concrete railroad tie (with somewhat disappointing results at the present time) shows a desire for a tie of this type. In parts of Europe where steel or wooden ties are readily obtained at reasonable cost, the concrete tie does not make much headway. In other parts where ties of

timber and steel are expensive, for instance in Italy, where the steel and wood tie is at least as expensive as the concrete, there has been considerable development and the officials informed me that more than 300,000 were in use in the Italian railways and that there were contracts for upwards of a million. The tie, however, has not a very great life when used in main line service, where it is subjected to the frequent passage of heavy locomotives. In what are termed secondary lines and sidings, ties

FIG. 6.—REINFORCED CONCRETE TIES IN MAIN LINE ITALIAN RAILWAY AT PORTO NACCIO, ITALY.

of reinforced concrete are reasonably effective and in Italy have as much as six or seven years of life.

I inspected some railroad ties just outside of Rome and found that these ties (see Fig. 6), which had been in service for about two years, were not wearing very well. A number of ties having crushed just inside the rail.

The method of fastening the tie consists of the use of a wooden block, cylindrical in shape, which is driven into the hole molded in the tie and an ordinary wood screw which fastens

the rail to the tie. When the threads in the block are worn by use, the block is replaced.

Fig. 7 is an illustration of the section of track of the London and Southeastern Railway at Knockholt station in which reinforced concrete ties have been used. These ties, I think, are not of as good design as those used in the Italian railways. They have been in service about two years. A number of them have failed in the manner shown in the illustration (Fig. 8) by the concrete crushing just inside the rail. It is my opinion that the difficulty in reinforced concrete ties is in a lack of proper analysis

FIG. 7.—REINFORCED CONCRETE TIES IN SECTION OF LONDON AND SOUTHEASTERN RAILWAY AT KNOCKHOLT STATION.

of the stresses, and that a tie could be so designed as to properly care for these stresses and thus prevent the breaking down of the tie in the manner just referred to. With this point cared for the life of the tie would be greatly prolonged.

It was the universal opinion of track men that through the use of reinforced concrete ties the cost of maintenance could be materially reduced and the alignment of the track much more readily maintained. It is, however, on curves that the ties are least effective and their life very brief. Another objection seems to be that the use of the concrete tie usually results in a rigidity

of roadbed which is extremely undesirable from an operating point of view, with the result that many devices have been tried with a view to introducing some elastic medium which will absorb the shock of passing locomotives.

At the Exposition in Turin were shown a number of reinforced concrete ties with wood blocks, fiber cushions, and other similar elastic shock absorbers, placed in the tie with a view to increasing its life. In most cases the design of the tie seemed

FIG. 8.—MANNER OF FAILURE OF REINFORCED CONCRETE TIES IN LONDON AND SOUTHEASTERN RAILWAY.

to be at fault; many of them had been developed by mechanics not skilled in structural designing, with the result that the reinforcement was not properly placed with regard to the stresses, especially those of impact; and it appears to me that the concrete tie problem can only be solved with a due consideration for these stresses.

Every one of the railroad officials who has had to do with concrete ties in Europe feels sure that a tie will be developed which will overcome the objections above indicated and reduce

the initial cost to a point where its life and reduced cost of maintenance will make it the cheapest railroad tie.

The objection in this country to the reinforced concrete tie, namely, its rigidity under moving loads and the consequent objectionable hammering to the locomotive, can be cared for by the introduction of an elastic cushion in the shape of fiber or wood. I believe the concrete tie will be the tie of the future and its first cost will prove of minor consideration when the reduced cost of maintenance and durability is considered.

Another interesting development in the use of concrete is in connection with sewers or conduits of circular or elliptical form in which the walls are made up of segments of concrete, which after being placed in position are grouted or cemented together, forming a solid ring. There were a number of cases where unusual economy had been effected through the use of these sewers. In some of them the segments were grooved along the axis and around the circumference in which the reinforcement was placed.

The use of reinforced concrete for the lining of sewers and tunnels, in my opinion, is a very important application. The possibility of molding these blocks and placing them in position, and filling the space between the roof and the ring with concrete forms a very simple and effective method of tunnel lining, the application of which is cheaper than brick and also cheaper than concrete construction where tight forms must be constructed and maintained in position until the concrete has properly hardened. In the segmental method, with the completion of the ring, the concrete backing may be readily placed in position and but little shoring will be necessary until the concrete has set.

Another extremely interesting matter was the use of reinforced concrete pipe of varying lengths which was laid as shown in Fig. 9 to conform to the general contour of the ground, the connections and adjustment being effected by means of loose sleeves slipped around the joints. After the pipes and sleeves are in position the spaces between the sleeves are grouted solidly to the pipe This application is in advance of anything we are doing in this country. Sections of reinforced concrete pipe laid in this manner, in lengths of 12 feet or more, would have great possibilities for use in pressure lines. The pipe in the illustration were made in a machine not unlike a lathe. The reinforcement

was placed around a core rotating horizontally and a very stiff mortar thrown against it. After the mortar had been built out to the requisite thickness, the pipe was wrapped with cotton bands four or five inches in width. The pipe was then removed and the wrapping kept wet until the mortar had set, when the

FIG. 9.—REINFORCED CONCRETE PIPE OF VARYING LENGTHS IN USE IN SWITZERLAND.

bands were pulled off. This method of construction naturally requires a great deal of labor and even if considered desirable would be entirely too expensive for use in this country.

Fig. 10 shows a cement products yard in Vienna and particularly illustrates the concrete ducts which are being used in

that city. These ducts have longitudinal recesses in which reinforcement is placed and the remaining space filled with cement mortar; this stiffens the ducts and holds them in position. It seemed to me there was an excellent opportunity for this type of construction in this country. When properly cemented in position, the ducts formed were in every way desirable and I am told that the cost is much less than that of terra cotta or other material.

During my last visit in Vienna I had occasion to refer to the use of concrete pavements and I find that this type is coming

FIG. 10.—CEMENT PRODUCTS PLANT OF PITTEL AND BRAUSEWETTER AND E. GAERTNER IN VIENNA, AUSTRIA.

into gradual use. The pavement inspected on my previous visit and reported to you two years ago had been laid four or five years and during my last visit I found these concrete roadways still in excellent condition and that they had not been repaired. The city of Vienna was engaged in laying considerable yardage of these pavements, especially around the Royal Palace. This pavement was of great width and was entirely of concrete. I am absolutely convinced of the durability of these pavements and repeat what I have stated in my previous address, that I believe they will come into general use.

The tank of reinforced concrete in use at the coal mines, in Rotherham, England, to remove the coal dust from the water coming from the coal washery is of considerable interest. This water is pumped into the tank, the coal dust is removed and the clean water returned to the washery. This coal dust is used in briquetting coal. Steel tanks are not available for this purpose because the sulphur in the coal would so corrode the steel as to make its life extremely short. The concrete tank has proved highly satisfactory not only in its resistance to the action of sulphur but in its water-tightness.

I commented on the artistic use of reinforced concrete in bridge construction in my previous address, but the matter is so striking that I cannot help referring to it again. The innum-

FIG. 11.—MEMORIAL REINFORCED CONCRETE BRIDGE CONNECTING HISTORICAL AND ART EXPOSITIONS OVER TIBER, IN ROME, ITALY.

erable bridges throughout Europe, carefully designed, carrying railroad as well as ordinary highway traffic, are monuments to the ability of the European engineer. The designs are for the most part graceful and show a wide diversity in artistic treatment.

A structure of great beauty is the Memorial Bridge (Fig. 11) in Rome connecting the Historical and Art Expositions, located on opposite sides of the Tiber. This bridge was built by the Hennebique Construction Company and is an example of European engineering skill. By reason of floods, it was necessary to construct the centering on which this bridge was built, of reinforced concrete—which is unusual and the first application of the use of reinforced concrete for centering that has come to my attention.

The bridge has a span of 100 meters and is built in imitation

of the native Travertine stone; the beauty of this is fully as great as the artificial Travertine stone to be found in the New York station of the Pennsylvania Railroad. This structure was built by the Italian government, and many severe tests were applied with but extremely slight deflections.

The manner in which this bridge was tested is also of interest. A commission was appointed to conduct the tests for the government. These tests consisted in moving heavy steam rollers and marching soldiers in a solid mass over the bridge; particularly interesting was the loading of the bridge solidly with soldiers and studying the effect of their cadenced step on the structure.

FIG. 12.—REINFORCED CONCRETE TRUSS 113.8 FT. SPAN USED IN MAIN RAILROAD STATION IN LEIPSIC, ERECTED FOR TESTING IN COSSEBAUDE, NEAR DRESDEN, GERMANY.

I think it may be stated without fear of contradiction that reinforced concrete structures properly designed are less affected by vibratory and cadenced loads than any other structure. It is particularly noticeable that in structures of identical design as to carrying capacity, those of steel show more movement than those of reinforced concrete.

A most interesting structure is the truss (Fig. 12) erected initially by the firm of Dyckerhoff and Widman, at their plant in Dresden. It was erected for testing the actual strength and justify-

ing the use of a truss of this type in the main railroad station in Leipsic. The arch was loaded in various ways and the deflections observed, and the tests were so satisfactory that it was successfully used in the structure. It is unusual in this country to go to the expense of erecting a structure of this kind and applying tests in order to satisfy the officials that it is amply safe. It would, however, be an effective means of preventing failures.

A matter that impressed me was the development of structures, many of them articulated, composed of separately molded parts, and the views herein presented exemplify some of the more remarkable of these structures. The practice of casting members and then pinning them together and encasing the connection in concrete is a development that seems to meet with favor, and while this may not appeal to many of the conservative engineers of this country, it seems to me that when this pin connection is of the same design as pin connected steel members, there is no reason why this type of structure, when properly designed, should not be more serviceable by reason of the concrete covering which makes it more durable than a structure of steel. The concrete trestle used in connection with the mine at Floreffe, Belgium, the lower portion of this is used for the county roadway and the upper portion for the handling of cars to the "tipple" from the coal mine, is an illustration of a remarkable development of this principle. I was informed that the parts of this structure were separately molded and afterward erected in place in a very short space of time and that the cost was considerably less than a similar structure of steel or of timber. There is a tendency towards systems requiring separately molded members, and it seems to me that such systems afford an opportunity for economy. The weak points in such structures are the joints, and with the same attention to these connections that is given to steel structures there is no reason why the joint in a concrete structure should not be stronger than the weakest part of the separate members.

A development of the Visintini system was to me somewhat surprising, since this system is very little used in this country; the early attempts to introduce it were unsuccessful, chiefly because of its cost as compared with that of other systems of reinforced concrete construction. In Europe it has a wide appli-

cation for buildings and bridges. Fig. 13 shows the bridge being erected in Copenhagen, Denmark, with girders of system Visintini. This bridge, erected in connection with the new station that the State Railways were building, carries a street with two lines of electric trams over the railroad. The ornamental portion is entirely of molded concrete which was not subsequently treated. Where labor is inexpensive the cost of forms is not so important as the cost of material. In this country, where conditions are just the reverse, this system proves uneconomical. The bridge consists of deep girders spanned by beams all of the Visintini system. The latter spaced solidly so as to form a slab, under the railroad tracks and 20 inches apart between the tracks. I

FIG. 13.—HIGHWAY BRIDGE OF REINFORCED CONCRETE OVER STATE RAILWAY TRACKS, COPENHAGEN, DENMARK.

think this bridge might be said to be a type of unit construction. Certainly the bridge, which was nearly completed, presented an extremely beautiful, ornamental appearance and far more desirable than a similar structure of steel. I believe that this method of construction, which is in a measure illustrated in the flat slab construction in use in railroad bridges in this country, possesses great possibilities in the matter of appearance and speed of erection. All the beams of the system could be molded and the structure then erected continuously, replacing perhaps an old structure, without interfering with the traffic.

An interesting type of girder construction is shown in Fig. 14, a highway bridge at Desna, Austria, which consists of a series of Visintini girders supported on piers of rather unusual construction.

The application of this system for arch ribs of a building is illustrated in (Fig. 15) the ceiling of a church erected in Aussig, Austria, and which I think is also an application of unit system. The lower flange of the rib of the arch is extended so as to support the Visintini beams which form the ceiling. Another type of structure which shows a trend in Europe that conservative structural engineers in this country might call dangerous is illustrated Fig. 16. This is a trussed bridge practically a development of the Visintini system, which forms a central span, with Visintini girders and beams used in the approaches.

FIG. 14.—REINFORCED CONCRETE HIGHWAY BRIDGE, DESNA, AUSTRIA.

The photograph, Fig. 17, shows the coal bunkers of the extensive plant of the city gas works in Vienna. The one on the left side has most of the forms removed and illustrates the generally pleasing character of the structure, which is notable in view of the fact that it must be massive, in order to carry a large quantity of coal. The attempt to render this structure pleasing and ornamental could well be emulated by our American designers. The exterior surface is dressed with pneumatic tools and a color effect has been obtained which is not unlike that attained in the construction of the Connecticut Avenue Bridge in Washington, D. C.

FIG. 15.—PORTION REINFORCED CONCRETE ROOF OF CHURCH, AUSSIG, AUSTRIA.

FIG. 16.—REINFORCED CONCRETE TRUSS HIGHWAY BRIDGE NEAR VIENNA, AUSTRIA.

We have, of course, many water towers in this country, but the thing that most impresses me in connection with those in European countries, is the thinness of the tank walls and the fact that they are constructed without the aid of waterproofing; the density of the concrete and the manner of reinforcing is sufficient to so distribute the cracks as to render them water-tight.

In connection with buildings abroad it would seem that more attention is given to the exterior and interior finish, even in factory buildings, than is given in this country; the skill used in

FIG. 17.—REINFORCED CONCRETE COAL BUNKERS, CITY GAS WORKS, VIENNA.

securing a pleasing finish, even considering the low-priced labor of Europe, is very rarely more expensive than the rough finish commonly used here. The tendency toward flat slabs and the elimination of as many beams as possible shows, I believe, an unmistakable turn, which is reflected in this country in the recent development of the flat slab type of construction. In many places the elimination of beams results in paneled effects through the use of girders between the columns and mortising and molding the connection between the slab and girder in such a way as to produce pleasing ornamental effects. There is also a ten-

dency to panel the slab itself, which relieves its flatness and adds no little to the beauty of the interior finish. The use of much higher ceilings than are commonly used in this country renders flat slabs unnecessary for the distribution of light. The effect of the paneling is somewhat that of the Roman barrel arch, and this latter type of construction may be seen in the ceiling decoration of many European structures.

An excellent exterior finish having the appearance of gray stone was to be seen in the Cigarette Paper Factory in Vienna. This structure had been erected many years and the weathering had been so uniform as not to in any way mar its beauty. This building was illustrated in my address describing my trip of two

FIG. 18.—TRADE SCHOOL OF REINFORCED CONCRETE, VIENNA, AUSTRIA.

years ago and is again referred to because after an interval of two years the exterior finish of the building remains unchanged. The entire ornamentation is of concrete and serves as an excellent illustration of the artistic possibilities of this material.

The Trade School in Vienna (Fig. 18) is also an excellent example of the artistic treatment of concrete. The paneling and ornamental work are most excellent in character, and for this reason do not call forth the criticism which once was so rife in this country, where the crude structures which we erected left much to be desired from the æsthetic point of view. It is frequently the practice in Europe, especially in Vienna, to cast monolithic walls and tool them afterwards, in a manner to pro-

duce the effect of stone. This, however, does not appeal to me so strongly as the structure in which there is no attempt to imitate stone, but where the material is used to stand for what it is, producing pleasing ornamentation and a surface uniform in color and texture and free from those stains and cracks which are frequently seen in structures of concrete in this country.

FIG. 19.—CITY HALL OF REINFORCED CONCRETE, NEAR BADEN, AUSTRIA.

Fig. 19 is a view of the City Hall at Weikersdorf near Baden, Austria. The entire wall, with ornamentation, is of concrete without any attempt to imitate other material, which I think illustrates that this is the proper way in which to use concrete and that when so used the results are much more effective than where it is used in imitation of other materials.

A favorite form of exterior decoration is to apply plaster

to the rough concrete and mold it in a manner to produce artistic effects.

The photograph of the Villa Figari in Genoa, Italy, shown in Fig. 20, is an illustration of the splendid possibilities in the artistic use of reinforced concrete. This graceful, beautiful structure needs no comment. The erection of this Villa involved a number of interesting problems in design which I think were met

FIG. 20.—VILLA FIGARI, GENOA, ITALY.

more successfully through the use of this material than would be possible through the use of any other. There are a number of plants in this country engaged in making ornamental concrete products of a very high order, and fully equal to some of the best work done in Europe, but the American designer does not at the present time seem to appreciate the adaptability of concrete for use in the ornamental structural portions of buildings; it seems to me that development along the lines illustrated in the Villa Figari opens a wonderfully promising field.

As the architect and designing engineer more fully realize the architectural possibilities of concrete and apply them in building construction with a due appreciation for the aesthetic, there will result such graceful structures as will entirely meet the criticisms and objections to the many cumbersome, ungraceful and unattractive structures which are being erected at the present time.

The desire to render even ordinary structures beautiful is

FIG. 21.—REINFORCED CONCRETE CAR BARNS OF CITY OF VIENNA, AUSTRIA.

particularly observable in Europe and it is, therefore, unusual to see even a mill building in which no attempt has been made to give it a proper finish. The eye for the beautiful is a matter of education and growth and I presume that in time we will develop similar tastes so that our mill buildings will be given artistic finishes. It should be borne in mind, however, that the cost of this work in Europe is considerably less than in this country, by reason of the cheap labor and because of the greater quantity that is done; they have acquired more skill and there

are a great many more skilled men capable of doing this class of work.

The car barns of the city of Vienna, shown in the illustration (Fig. 21), is another development in the use of concrete which I think will become general in the future. This structure is built entirely of concrete, including the roof which has not been waterproofed. I understand that the cost of this structure was materially less than the cost of other types of construction. The span and lines of the girder was a matter of interest and is characteristic of the skill of the Austrian engineer.

A building of reinforced concrete which was brought to my attention during my visit two years ago, and which was then under construction, is the Urania Building in Vienna, which is devoted to the development of popular musical education. It has a rather difficult problem in acoustics which is of moment to those interested in concrete. The main auditorium has a floor and ceiling of reinforced concrete of considerable thickness; above and below this auditorium are smaller halls for musical entertainment. The large auditorium contains an organ, and there is no connection between it and the halls above and below, yet the Director states that when there is a concert in progress in the large auditorium it is impossible to hold a concert in either of the other auditoriums, because the sound of the music in the large hall is so pronounced as to seriously interfere with the performance in the other halls. This subject has received a great deal of consideration in Vienna and methods are now in progress to eliminate, if possible, by some insulating medium, the transmission of sound.

Another matter of interest was the work which the Austrian Association is doing in investigations and tests, not unlike those of our own Committee on Reinforced Concrete, except that the value of the work accomplished is greater because the work is much more extensive. They have conducted a comprehensive series of tests of columns and beams, the cost of the work being defrayed by the Austrian cement manufacturers, and has been under the auspices of representatives of the government, the Society of Engineers and Architects, the Austrian Concrete Association and the Austrian Association of Cement Manufacturers. The principal feature of the tests which were being conducted at

the time of my visit was a study of the effect of the weight of wall in restraining the ends of beams; a number of different methods were being tried and it had been found that the stiff-

FIG. 22.—A RESTRAINED REINFORCED CONCRETE BEAM AFTER TESTING.

FIG. 23.—METHOD OF APPLYING LOAD IN TESTING REINFORCED CONCRETE BEAMS.

ness of the beam was materially increased through the weight of the wall.

Fig. 22 illustrates one of these beams after test. In some

cases the beam rested on a pier, and in others it extended over; in still others it was imbedded in concrete, or laid up in a brick wall so as to approximate practical conditions.

Fig. 23 shows the manner of loading and observing the deflections. The method of applying the load is perhaps of interest. The load to be applied was carried by hydraulic jacks, the lowering of which brought the weight of this super-imposed load on the beam.

One thing that was apparent on my recent trip, and which must be a matter of great gratification to every member, is the high regard in which this Association and its work are held in Europe. It is taken generally as a model, and in many cases the character of the work done here and the value of its proceedings and discussions are so appreciated as to result in a number of Europeans becoming members in order to secure its publications.

There is also a cordial feeling of cooperative good-will between the various concrete organizations of Europe and our Association, which I hope may be fostered and that it may be possible through the interchange of delegates, papers, and in other ways, to extend this cordiality so as to obtain the advantages of the development in the art of concrete construction in various parts of the world.

This Association stands for education in the development of the proper use of concrete, and certainly it should be a part of its policy to encourage international co-operation, to the end that the development in this country may proceed with a full knowledge of what is being done by our foreign competitors.

REPORT OF THE COMMITTEE ON REINFORCED CONCRETE AND BUILDING LAWS.

In accordance with the program of tests on completed structures presented to the Association at the last convention, four tests have been made, two of them on standard forms of reinforced concrete floors in which the concrete slab is supported by one or more intermediate beams which in turn are framed into or supported by girders carrying the load to the columns. Of the other two tests one is on the girderless or flat slab floor, and the other on a combination of tile and concrete floor, in which the reinforcement is placed in two directions.

Making arrangements for the tests and computing the results have taken up all the available time of the Committee; the test data is therefore presented to the Association without any attempt at generalization.

The tests on the Wenalden and the Turner-Carter buildings were made under the direct supervision of A. N. Talbot; the other tests were made by F. J. Trelease of the Research Department of the Corrugated Bar Company, with the assistance of W. A. Slater of the Illinois Engineering Experiment Station.

Part I. Tests of Two Reinforced Concrete Buildings of the Beam and Girder Type.

Preliminary.—These tests were undertaken for the purpose of obtaining information on the action of the composite structure of concrete and steel under load in a reinforced concrete building constructed under the usual conditions of work. Many tests have been made of separate reinforced concrete members, but little attention has been given to the measurement of stresses and deformations in the completed building and to the determination of their actual amount and distribution and of the effect of one part of the structure upon another. Load-deflection tests are common and are of value in judging of the quality of workmanship and in giving confidence in the structure, but they throw little light on the stresses developed in the different parts or

upon their distribution. A variety of views have been advanced on the relation between the bending moment at a section at the support and that at the middle of the beam, on the amount of arch action which may be developed in the structure, on the distribution of the stresses across a floor slab acting as the flange of a T-beam, on the restraint of girders and beams, etc. These tests were undertaken in an effort to obtain some information

FIG. 1.—INSTRUMENTS AND TOOLS.

on such matters as well as to find something of the general action of reinforced concrete structures as a whole.

The general method of test followed the plan outlined by Arthur R. Lord in the paper, *A Test of a Flat Slab Floor in a Reinforced Concrete Building,** presented at the New York Convention. Holes were cut in the concrete until the reinforcing bars were bared. Gauge holes were then drilled in these bars, at distances apart to give the proper gauge lengths. Where measurements of deformation of the concrete were desired, holes were cut in the concrete and a metal plug inserted in which the

* See *Proceedings*, Vol. VII, p. 156.

gauge holes were later drilled. These gauge lines were selected in places where it was thought that critical stresses would be determined. In some places for one reason or another the reinforcing bars were inaccessible and it was impracticable to obtain measurements to give information which would have been of interest. In some cases a series of gauge lines were used to de-

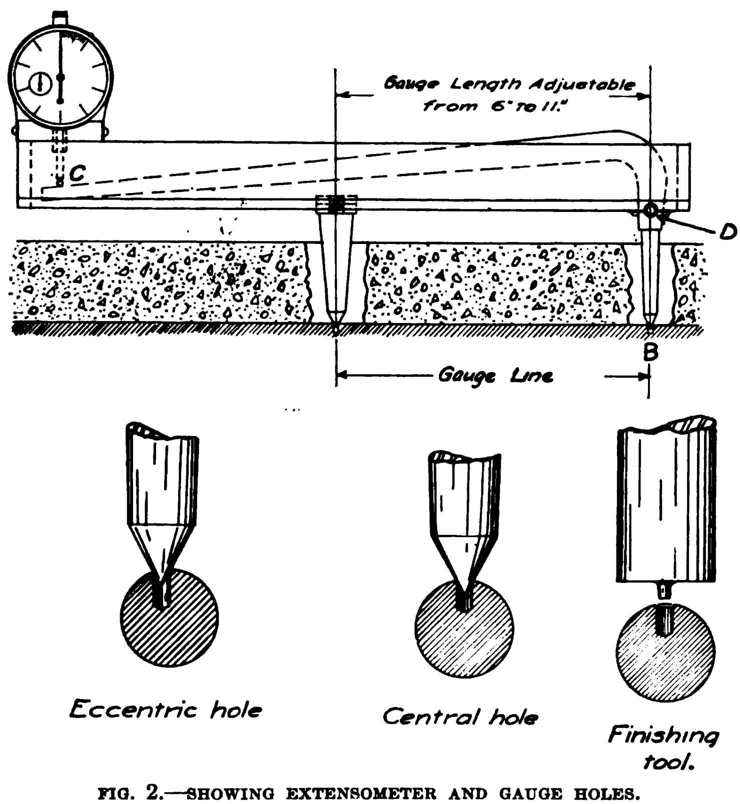


FIG. 2.—SHOWING EXTENSOMETER AND GAUGE HOLES.

termine the change of stress or distribution from one point to another as at the end of a restrained beam and across the floor slab between beams.

The measurements were made by means of Berry extensometers of the form developed at the University of Illinois. The extensometer is shown at the bottom of Fig. 1. The instrument

reads to $\frac{1}{1000}$ in. and is estimated to $\frac{1}{10000}$ in. Its make-up is shown in Fig. 2, as are the gauge holes. In making a measurement the legs of the instrument are inserted in the gauge holes, a reading taken, the instrument taken out and again inserted and read, and this proceeding repeated until a number of readings without serious discrepancies are found. The operation of making a

FIG. 3.—TAKING AN OBSERVATION.

measurement is shown in Figs. 3 and 4. The instrument is of a simple character, but its use requires unusual care and skill on the part of the manipulator. The method of using the instrument as well as the necessary general conditions attending such tests are comprehensively discussed in the paper presented at this convention by W. A. Slater on *Tests of Reinforced Concrete Buildings under Load.**

Acknowledgment.—These tests were undertaken through the

* See page 168.—ED.

efforts of the Committee on Reinforced Concrete and Building Laws of the Association, in co-operation with the Engineering Experiment Station of the University of Illinois. The money to defray the expense of the test was arranged for by the President and Treasurer of the Association. The contractors for the two buildings co-operated in the tests. The technical part of making the tests was done by members of the staff of the Engineering Experiment Station of the University of Illinois.

FIG. 4.—TAKING AN OBSERVATION.

Comments.—A few words on the basis and limitations of such tests may not be out of place. The measurements and observations are subject to some uncertainty; they are not exact or precise—some erratic readings must be expected. The measuring instrument is used under unfavorable conditions. The gauge holes are deep in the concrete and the measurements may be interfered with by dust or other obstructing matter. Great care and much skill is necessary in making observations. Each test of this kind made has shown advances in accuracy and certainty,

and further experience ought to show further progress. It must be understood that the structure itself is not entirely homogeneous and that all parts of it do not act alike. Further, the structure itself is tied together so closely that stress in one portion may be modified or assisted by another portion which may not be thought to affect it, and this in an unknown amount. The modulus of elasticity of the concrete in the structure may not be known. The load-deformation lines may be irregular and imperfect. This

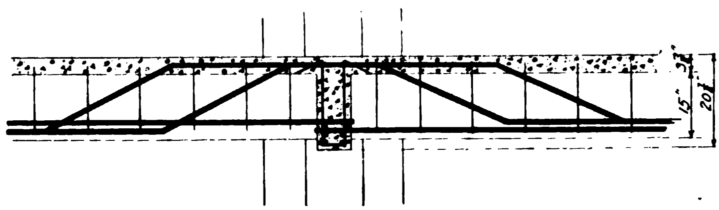
FIG. 5.—THE WENALDEN BUILDING.

all means that care must be taken in the interpretation of results. Important information will be brought out by such tests, as these tests show, and tests of special features of construction and an accumulation of data on the action of the structure as a whole will be worth many times the cost of the work.

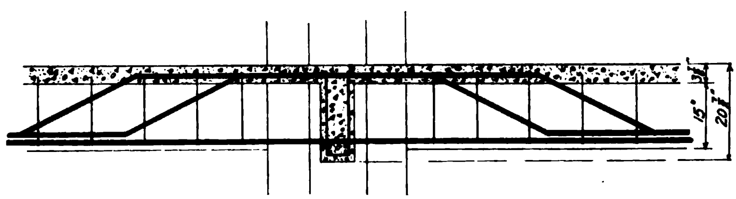
WENALDEN BUILDING TEST.

Building.—The Wenalden Building, Fig. 5, is a ten-story reinforced concrete structure at 18th and Lumber Streets, Chicago.

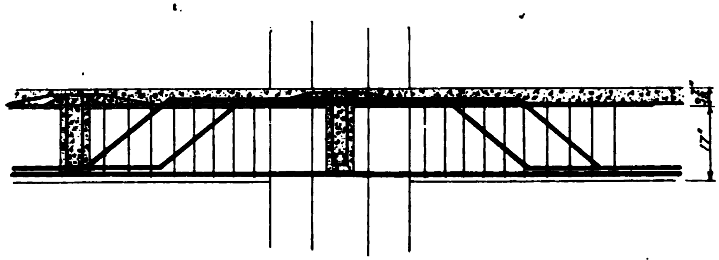
It was built by the Ferro-Concrete Construction Company, Cincinnati, Ohio, in accordance with the plans and specifications of Howard Chapman, architect. It is now occupied by Carson, Pirie, Scott and Company, dry goods merchants, as a warehouse.



Elevation of Intermediate Beam.



Elevation of Column Beam.



View of Girder.

FIG. 6.—GENERAL POSITION OF REINFORCEMENT.

The building is of the beam and girder type. The floor panels are 15 ft. by 20 ft. The girders are placed between columns in the short direction. Floor beams extend the long way of the panel, there being two intermediate beams built into and supported by the girders and a column beam built into and supported by the

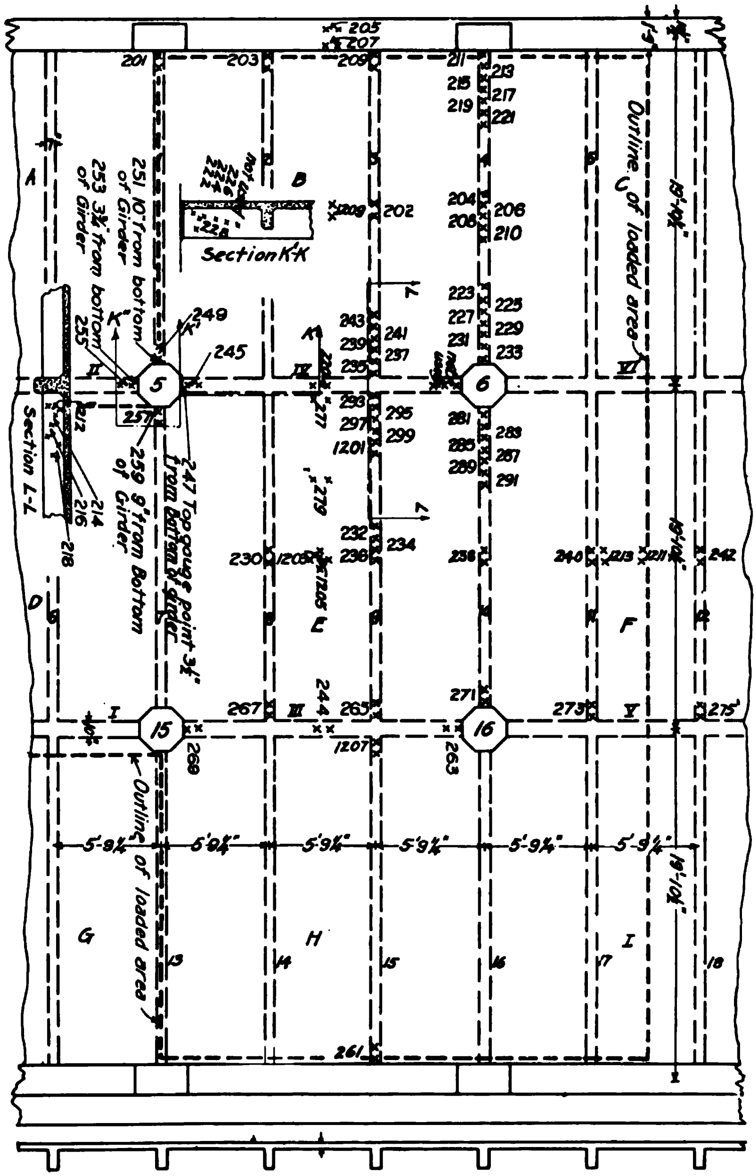


FIG. 17.—PLAN SHOWING LOCATION OF GAUGE LINES ON UNDER SIDE OF FLOOR.

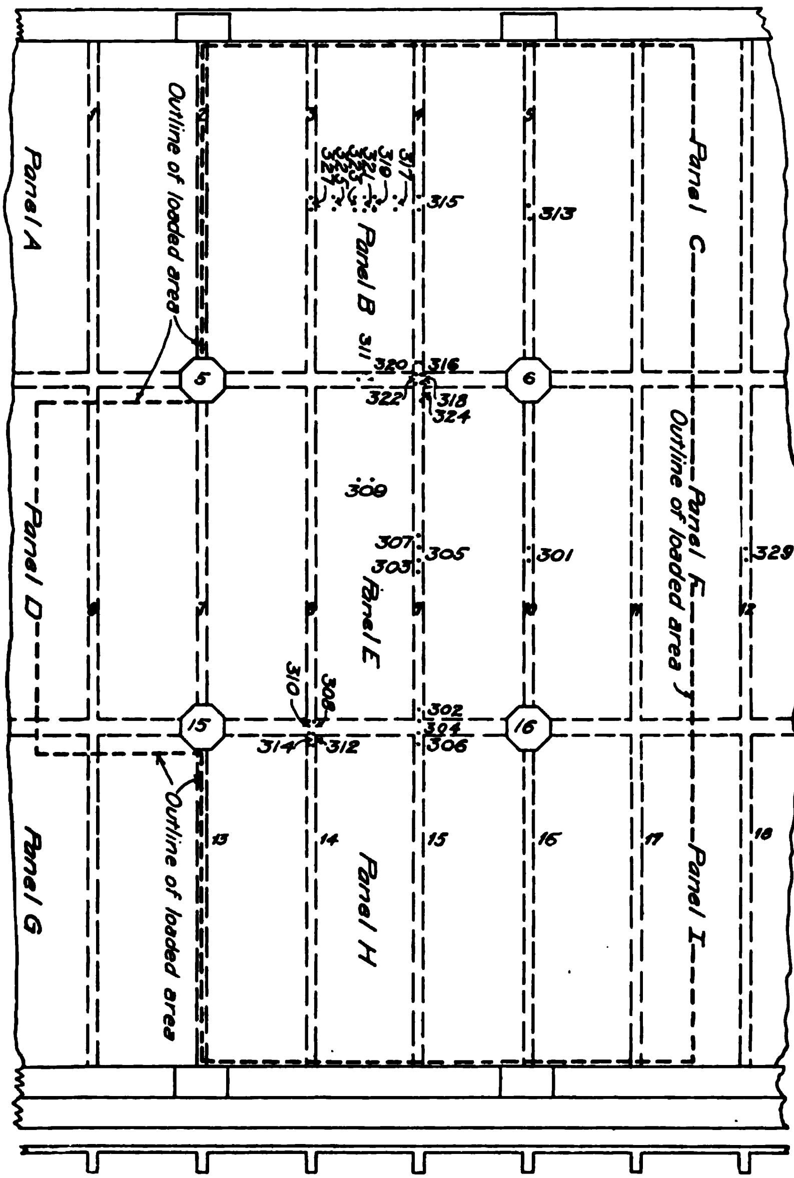


FIG 18.—PLAN SHOWING LOCATION OF GAUGE LINES ON UPPER SIDE OF FLOOR.

Preparation for the Test.—A week was used in preparing for the test. Platforms supported by scaffolding for the use of observers were built on the second floor. Independent of this was a framework, which was supported by the second floor, for use in making measurement of deflections. The boxes for holding the sand were constructed, this being facilitated by a power saw located on the second floor. Considerable time was consumed in drilling holes in the concrete to bare the reinforcement.

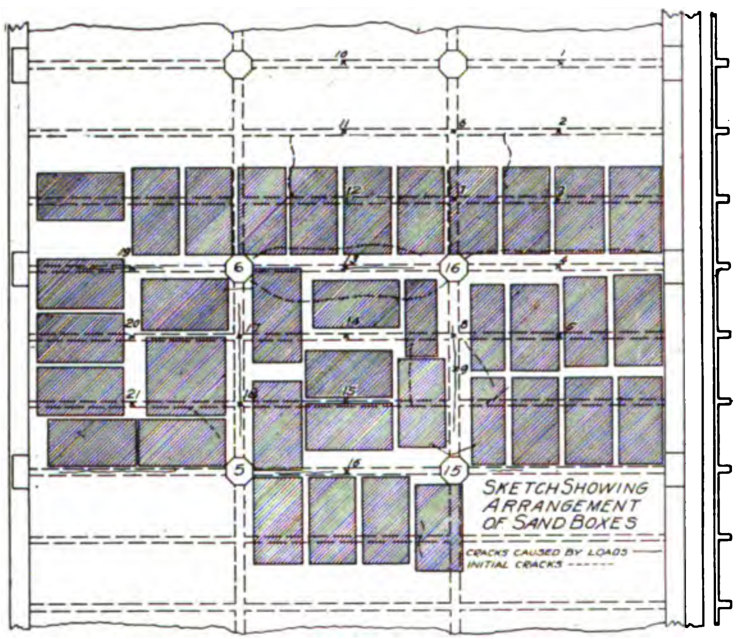


FIG. 19.—LOCATION OF SAND BOXES AND FLOOR CRACKS.

In some cases this was found to be at a considerable depth from the surface. In all nearly two hundred holes were cut in the concrete. Holes were drilled in the reinforcing bars, as heretofore described, for use as gauge points. The gauge length was made 8 in. The position of the gauge lines for the reinforcing bars is shown on Figs. 17 and 18 by the even numbers. For use in the measurement of deformations of the concrete, holes about $\frac{1}{2}$ in. in diameter and 1 in. deep were drilled in the concrete and steel

FIG. 20.—VIEW OF SAND BOXES.

FIG. 21.—VIEW OF TEST LOAD IN TURNER-CARTER BUILDING.

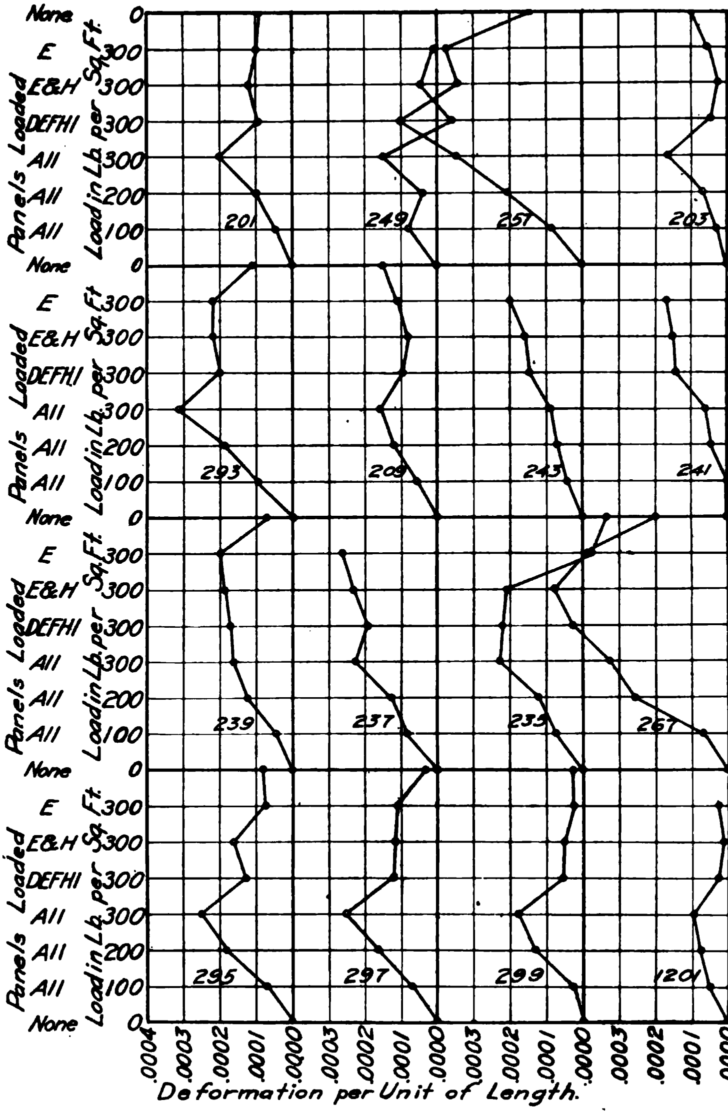


FIG. 22.—LOAD DEFORMATION DIAGRAMS FOR UNDER SIDE OF BEAMS AT END.

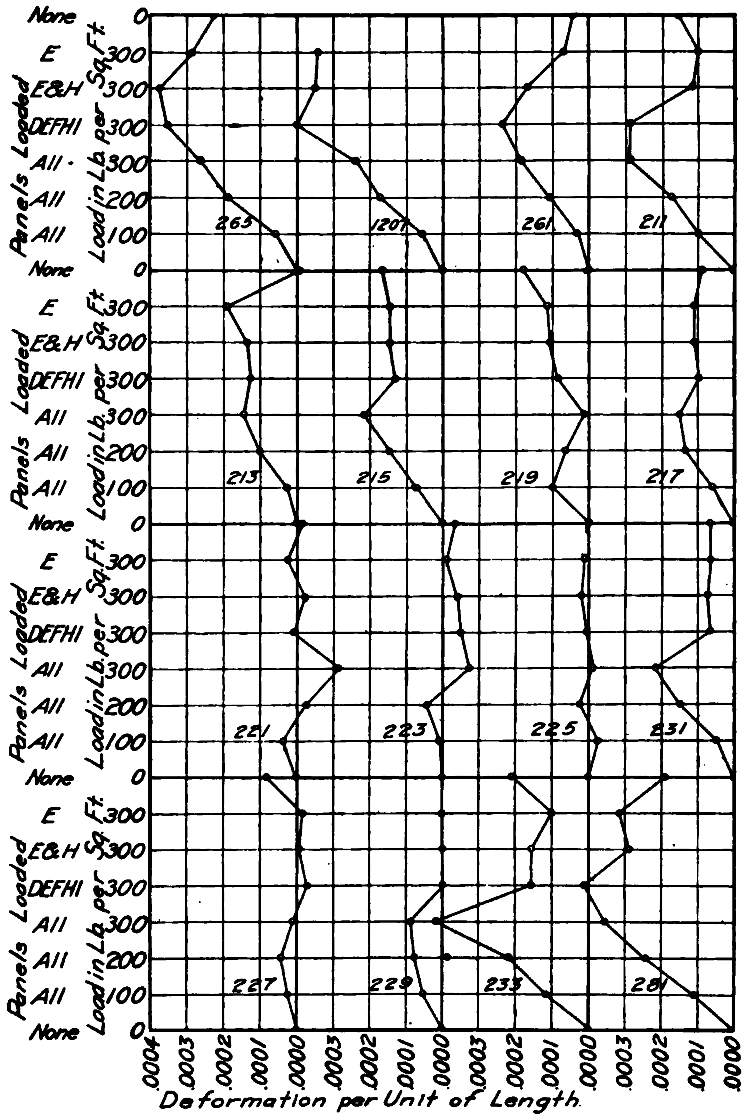


FIG. 23.—LOAD DEFORMATION DIAGRAMS FOR UNDER SIDE OF BEAMS AT END.

plugs were inserted and set in plaster of Paris. Gauge holes for receiving the points of the extensometers were drilled in these plugs with a No. 54 drill. The position of the gauge lines is shown in Figs. 17 and 18 by the odd numbers. The gauge length was 8 in.

The deflections were measured between a steel ball set in the under surface of the beam and a ball attached to the framework previously described. The measurements were made by means of the micrometer shown in Fig. 1.

Method of Loading.—The test area was on the third floor. The loading material was damp sand which was placed in bottomless boxes. These boxes were of various sizes and were placed in such a way as to give a well distributed load. The general size of the box was 4 ft. 6 in. wide, 8 ft. long and 4 ft. 6 in. deep. Fig. 19 shows the position of the boxes and the test area. Fig. 20 is a view with the sand boxes ready for loading. The boxes were made small enough to permit a good distribution of load even though part of the weight of the sand might be carried by arching and friction down the sides. The test area covered three full panels and parts of four others, in all equivalent to five panels. A loading space was chosen which it was thought would give the fullest stresses over the girders and beams on which the principal measurements were made. In removing the load the outer panels were unloaded first in an attempt to determine the relation between single panel loading and group loading. The load applied was the equivalent of 300 lb. per sq. ft., double the design live load.

Before beginning the test, a calibration of the heaviness of the sand was made by weighing the sand which had been shoveled into a box of 16 cu. ft. capacity placed on the scales. It was found that there was a difference of about 10 per cent. in the weight of sand which had been thrown in loosely and sand which was packed somewhat. During unloading, the entire contents of three of the sand boxes (about 500 cu. ft.) were weighed. This gave an average of 88.6 lb. per cu. ft., agreeing closely with the weights of the unpacked sand previously weighed, and this value was used in the calculation of loads.

On a part of the area where the boxes were not carried to a sufficient height and where the space was not covered adequately by them, cement in sacks was used as loading material.

The supply of sand for the loading had previously been delivered on the same floor, the piles being kept at least one panel away from the location of the test area, and this was distributed over sufficient floor space that the stresses in the beams of the test area could not be affected. In applying the load the sand was wheeled in barrows and dumped into the boxes. As the sand was placed, the sides of the boxes were rapped to break the adhesion of the sand. Some leveling of the sand in the boxes

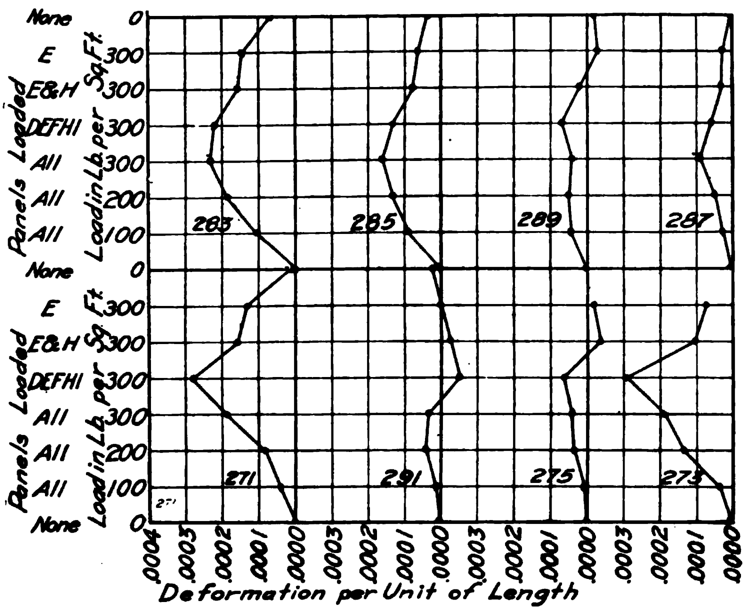


FIG. 24.—LOAD DEFORMATION DIAGRAMS FOR UNDER SIDE OF BEAMS AT END.

was done, but there was little compacting by tramping or otherwise.

Making the Test.—A very important element of a test of this kind is the initial observation for fixing the zero point of the test readings. Three sets of observations for a number of gauge lines were made before the beginning of the test, on the afternoon of September 10 and the forenoon of September 11. Where discrepancies were found new observations were made. Even with this number of observations there are uncertainties

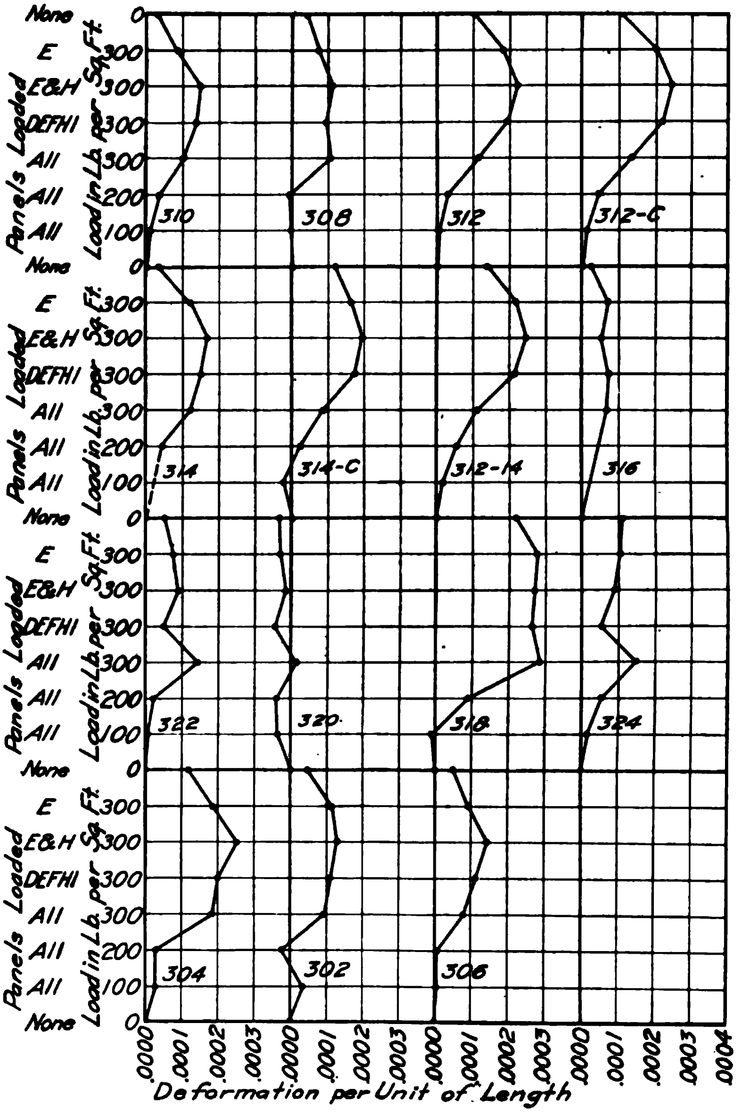


FIG. 25.—LOAD DEFORMATION DIAGRAMS FOR UPPER SIDE OF BEAMS AT END.

in some initial readings. Experience confirms the view that before any load is placed the initial readings which have been taken should be worked up and observations repeated until all discrepancies and uncertainties have been removed.

Readings were taken immediately after the completion of each

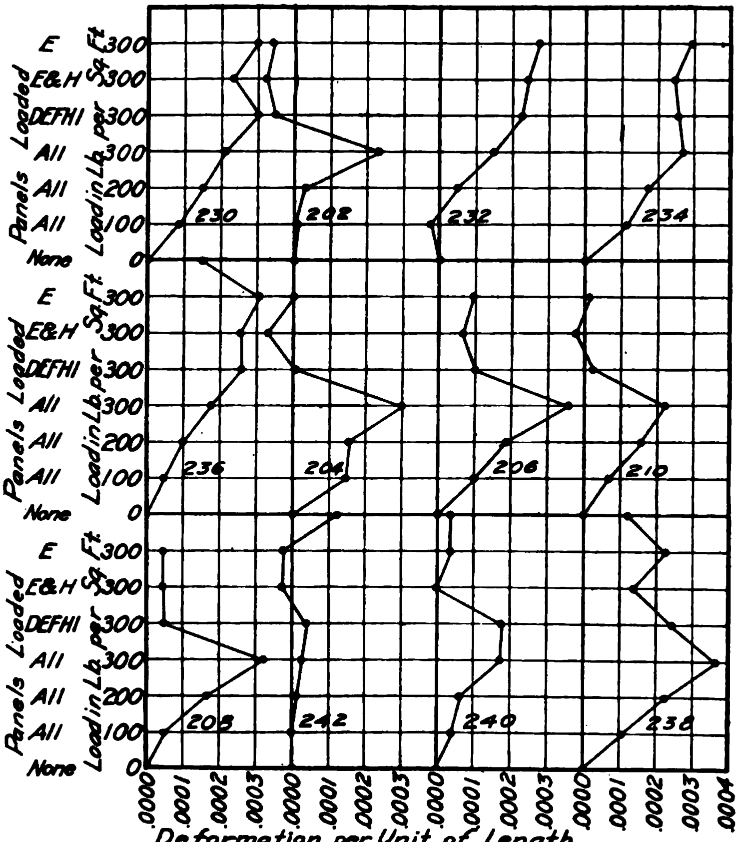


FIG. 26.—LOAD DEFORMATION DIAGRAMS FOR UNDER SIDE OF BEAMS AT MIDDLE.

increment of load and again immediately before the beginning of placing another increment of load. This usually corresponded with evening readings and morning readings. A series of readings was also taken with the full test load on. These extended over a

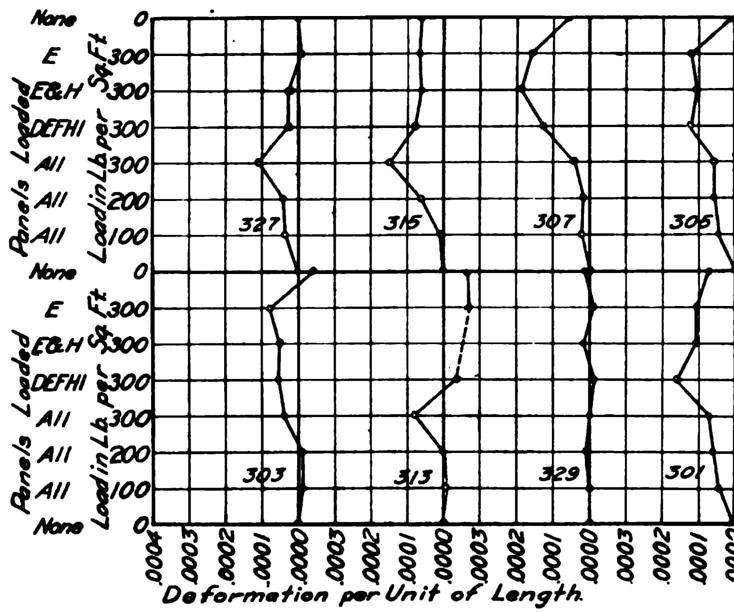


FIG. 27.—LOAD DEFORMATION DIAGRAMS FOR UPPER SIDE OF BEAMS AT MIDDLE.

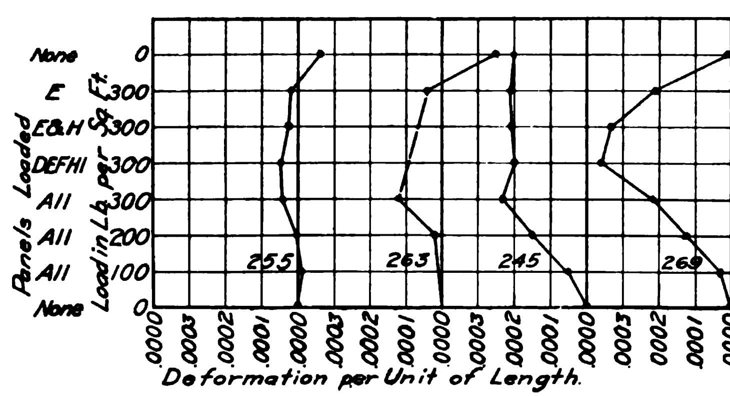


FIG. 28.—LOAD DEFORMATION DIAGRAMS FOR UNDER SIDE OF GIRDERS AT END.

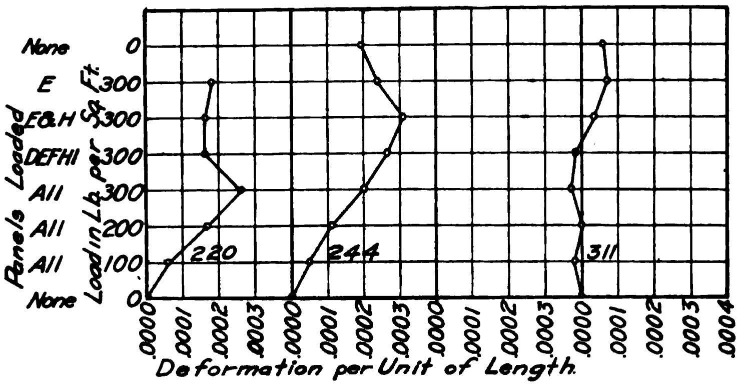


FIG. 29.—LOAD DEFORMATION DIAGRAMS FOR UPPER SIDE AND UNDER SIDE OF GIRDERS AT MIDDLE.

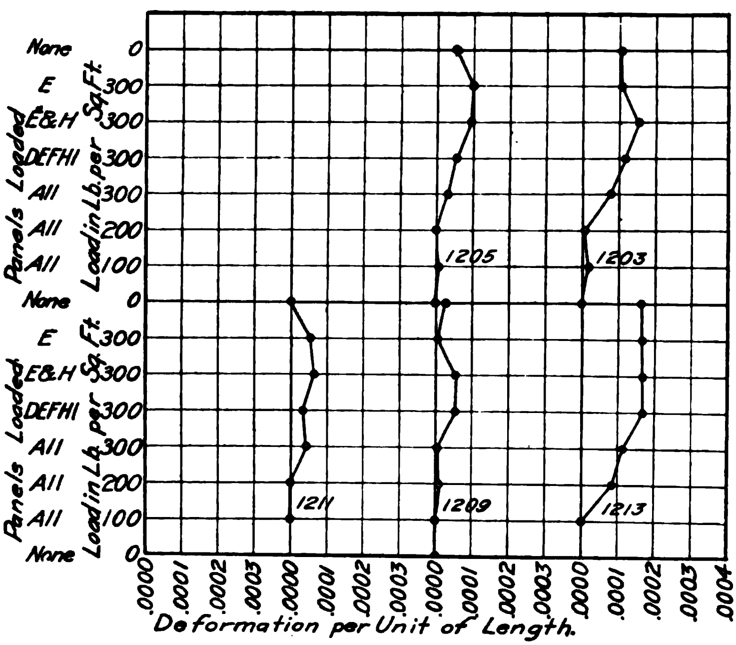


FIG. 30.—LOAD DEFORMATION DIAGRAMS FOR CONCRETE ON UNDER SIDE OF SLAB.

TABLE IV.—SCHEDULE OF LOADING OPERATIONS IN TURNER-CARTER BUILDING TEST

Loading Schedule.

Day.	Date.	Observations.		Loading.		Observations.	
		Load, lb. per sq. ft.	Hours.	lb. per sq. ft.	Hours.	Load, lb. per sq. ft.	Hours.
Sunday......	9-10-11	0	12 M. to 2 P. M.				
Monday.....	9-11-11	0	7.20 to 12 M.	100	1.30 to 6.00 P. M.	100	6.10 to 8.00 P. M.
Tuesday.....	9-12-11	100	6.30 to 8.15 A. M.	200	10.30 A. M. to 3.00 P. M.	200	3.10 to 5.30 P. M.
Wednesday..	9-13-11	200	6.20 to 8.20 A. M.	300	9.00 A. M. to 3.30 P. M.	300 300	3.50 to 5.50 P. M. 10.30 to 11.30 P. M.
Thursday....	9-14-11	300	8.00 to 8.30 A. M.			300	3.00 to 3.30 P. M.

Unloading Schedule.

Day.	Date.	Observations.		Loading.		Observations.	
		Load, lb. per sq. ft.	Hours.	lb. per sq. ft.	Hours.	Load, lb. per sq. ft.	Hours.
Friday......	9-15-11	300	7.30 to 9.30 A. M.	300 on *D*, *E*, *F*, *H*, and *I*.	3.30 to 7.30 P. M.	300 on *D*, *E*, *F*, *H*, and *I*.	8.00 to 8.30 P. M.
Saturday....	9-16-11	300 on *D*, *E*, *F*, *H*, and *I*.	7.20 to 9.15 A. M.	300 on *E* and *H*.	9.30 to 11.45 A. M.	300 on *E* and *H*.	6.30 to 8.00 P. M.
Monday.....	9-18-11	300 on *E* and *H*.	6.15 A. M. to 9.20 A. M.	300 on *E* only.	9.30 A. M. to 12.00 M.	300 on *E* only.	12.15 to 1.50 P. M. 4.15 to 8.00 P. M.
Tuesday.....	9-19-11					300 on *E* only.	4.50 to 6.50 P. M.
Wednesday..	9-20-11	300 on *E* only.	8.30 A. M. to 12.30 P. M.	Zero.	1.00 to 3.40 P. M.	Zero on all bays.	4.00 to 5.40 P. M.

period of 48 hours. A similar method was used in the process of removing the load.

Table IV shows the loading schedule. The load was applied in increments of 100 lb. per sq. ft. based upon the whole test area. The application of the load consumed three days. The full load was left on 48 hours. The unloading schedule is also shown in Table IV. In the unloading, the load on panels *B* and *C* were first removed, then the load on panels *D*, *F*, and *I*, followed by the

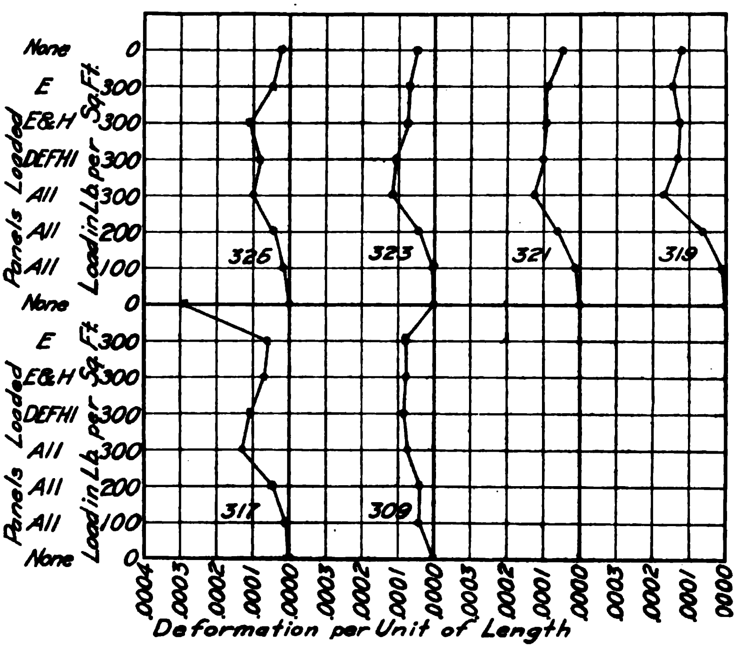


FIG. 31 —LOAD DEFORMATION DIAGRAMS FOR CONCRETE ON UPPER SIDE OF SLAB.

removal of the load on panel *H*. Fig. 21 is a view at a load of 300 lb. per sq. ft. over the test area. The total load was over 500,000 lb.

Personnel of Testing Staff.—All instrument readings were made by W. A. Slater and H. F. Moore, of the staff of the Engineering Experiment Station of the University of Illinois. Mr. Slater had immediate charge of the test as a whole. A. N. Talbot was present during the work of preparing for the test. Three others assisted in the work of recording and reducing data.

TABLE V.—STRESS INDICATIONS IN TURNER-CARTER BUILDING TEST.

Member.	Gauge Line.	Reinforcement.	Gauge Line.	Concrete.
End of girder.	. . .	. . . . . .	269	900
Middle of girder	220	8,000	311	Little
"	244	9,000	. . .	. . . . . .
End of beam	304	8,000	265	1,100
"	318	8,000	267	1,100
"	310	4,000	281	1,000
"	. . .	. . . . . .	293	800
Middle of beam.	202	7,000	301	350
"	206	11,000	305	350
"	230	9,000	313	200
"	234	8,000	315	300
"	236	8,000	. . .	. . . . . .
"	238	11,000	. . .	. . . . . .
"	240	5,000	. . .	. . . . . .
Bent up bar in girder.	222	5,000	. . .	. . . . . .
"	224	5,000	. . .	. . . . . .
Bent up bar in beam.	214	—3,000	. . .	. . . . . .

TABLE VI.—MAXIMUM STRESSES AND MOMENT COEFFICIENTS IN TURNER-CARTER BUILDING TEST.

Member.	Reinforcement.		Concrete.	
	Stress.	Coefficient.	Stress.	Coefficient.
Girder, End.	*31,000*	*1/12*	*1,200*	*1/12*
" End.	. . . . . .	. .	900	0.06
" Middle	*12,500*	*1/12*	*300*	*1/12*
" Middle.	8,000	0.05	Little	
Intermediate Beam, End.	*21,500*	*1/12*	*1,300*	*1/12*
" End.	8,000	0.03	1,100	0.07
" Middle.	*18,500*	*1/12*	*380*	*1/12*
" Middle.	11,000	0.05	350	0.077
Column Beam, End.	*10,600*	*1/12*	*1,200*	*1/12*
" End.	. . . . . .	. .	950	0.064
" Middle.	*17,000*	*1/12*	*350*	*1/12*
" Middle.	10,000	0.05	225	0.054

Deformations and Stresses.—The results of observations on various gauge lines for the beams and girders are plotted in Figs. 22 to 29. Fig. 30 gives the deformations in the concrete on the under side of the floor slab and Fig. 31 those on the upper side Fig. 32 records measurements made on the bent-up bars and stirrups.

As already stated, the location of the gauge lines is shown on Figs. 17 and 18, the odd numbers referring to measurement on the

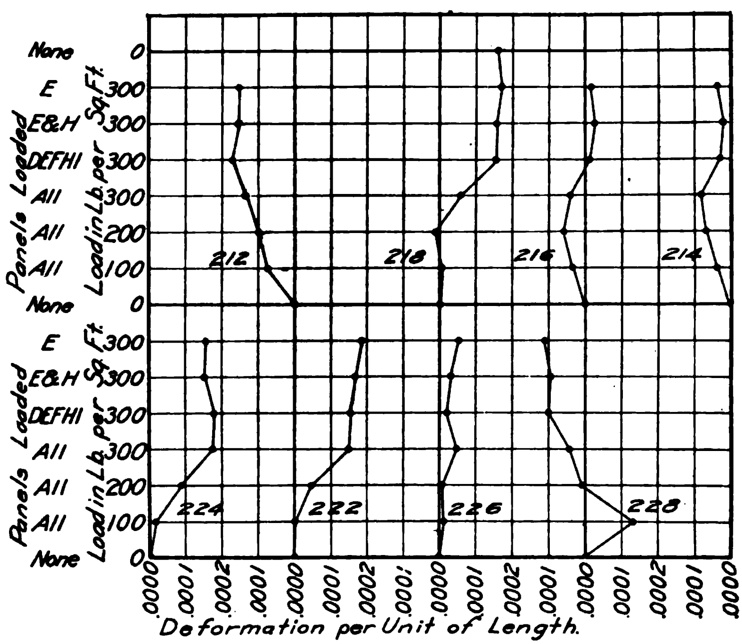


FIG. 32.—LOAD DEFORMATION DIAGRAMS FOR BENT-UP BARS AND STIRRUPS.

concrete, the even numbers to measurement on the reinforcement. The numbers in the two hundreds are gauge lines on the under side or second story side, and the numbers in the three hundreds are on the upper side or third story side.

Stresses and bending moment coefficients are tabulated in Tables V and VI.

The suggestions given for caution and care in interpreting measurements should be applied to this test.

Beams.—For the tensile stresses in the reinforcement at the middle of the intermediate beams at the full load of 300 lb. per sq. ft., the highest stress observed was 11,000 lb. per sq. in. and the average stress recorded may be said to be 8,500 lb. per sq. in. At the ends of the intermediate beams, the highest stress observed in the reinforcement was 8,000 lb. per sq. in., and the general value may be said to be 7,500 lb. per sq. in. Using the assumptions for resisting moment ordinarily taken in design calculations, these stresses may be considered to correspond to a bending

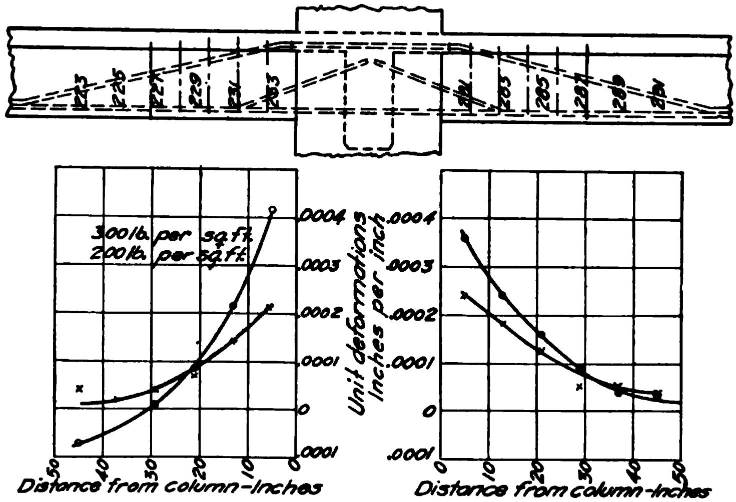


FIG. 33.—DIAGRAM SHOWING DISTRIBUTION OF COMPRESSIVE DEFORMATION IN BOTTOM OF COLUMN BEAM.

moment coefficient of .05 Wl for the maximum stress at the middle of the beam and .03 Wl for the maximum stress at the end of the beam, if the tensile strength of the concrete be not considered.

Assuming a modulus of elasticity for the concrete of 2,500,000 lb. per sq. in., the concrete on the compression side of the beams at the middle showed a compressive stress of 350 lb. per sq. in. and at the end of the beam 1,100 lb. per sq. in. It is apparent that the total compressive stress in the concrete is greater than the total tensile stress in the reinforcement of the beams. A possible explanation is that end thrust exists, involving so-called

arch action in the beams and floor structure, and that the tensile stress is relieved by the presence of this thrust. The tensile strength of the concrete must have a large effect on the resisting moment. The coefficient of Wl in the bending moment, necessary to give a compressive stress equal to the maximum measured in the concrete, on the assumptions made, is .077 for the middle of the beam and .07 for the end of the beam. These coefficients are lower than the value 1/12 usually assumed in design of such beams.

Girders.—For the tensile stresses at the middle of the girders the observations showed about 8,000 lb. per sq. in. in the reinforce-

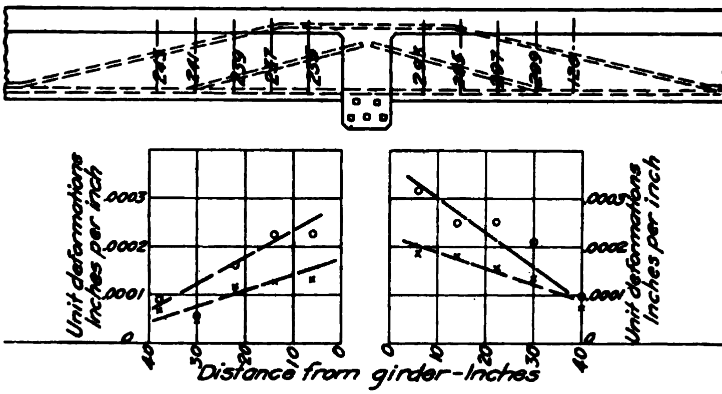


FIG 34.—DIAGRAM SHOWING DISTRIBUTION OF COMPRESSIVE DEFORMATION IN INTERMEDIATE BEAM.

ment at the middle. This corresponds to a bending moment coefficient of .05, again neglecting the tensile strength of the concrete. The reinforcement at the end of the girder was inaccessible.

Assuming a modulus of elasticity of 2,500,000 lb. per sq. in., the concrete on the compressive side of the beam at the support showed a compressive stress of 900 lb. per sq. in. The reading at the middle of the beam showed very little compression. Assuming that the loads on the girder are concentrated at the points where the intermediate beams are connected, and making the same assumption of distribution of stress as before, the coefficient of bending moment was .06. It seems probable that the compression at the middle of the span must be distributed over a consid-

erable width of floor, or larger readings of compression would have been obtained.

Decrease in Compression with Distance from Support.—In four beams measurements of compressive deformations were taken at a series of gauge lines from the support to a location near the point of inflection. The position of these points is shown in Fig. 17. The gauge lines No. 223, 225, 227, 229, 231, and 233 are on one side of column No. 6, and 281, 283, 285, 287, 289, and 291 are on the other side of column No. 6. It may be expected that there will be full restraint for the end of the beams. Gauge lines 243, 241, 239, 237, and 235 are on one side of a girder and 293, 295, 297, 299, and 1201 are on the other side. The unit-deformations for these gauge lines at loads of 200 lb. per sq. ft. and 300 lb. per sq. ft. are plotted in Figs. 33 and 34.

The measurements recorded for the column beams show considerably more compressive stress than do those for the intermediate beams, perhaps one-third more. This difference in stress may be due partly to the deflection of the girder, and to the deflection of the intermediate beam between its support and a point opposite the end of the column beam, which would permit a larger part of the load to be carried by the column beam. It may be due somewhat to the fact that reinforcing bars are bent down from a point at the end of the column beam, while in the intermediate beams the bars run horizontally for a foot from the face of the girder.

The direction of the lines in Fig. 33 and Fig. 34 indicates a zero stress at about 45 in. from the face of column in the column beams and at about 50 in. from the face of the girder in the intermediate beams. In both cases the results locate the point of inflection at about 0.22 of the clear span.

T-Beam Action.—The distribution of compressive stresses in the T-beam formed by a beam and the floor slab (which involves the distances away from the beam for which compressive stresses are developed) has been a fruitful source of discussion. Measurements parallel to the axis of the beam were taken on the upper surface of the floor slab immediately above beams and at intervals between them. These gauge lines are No. 315, 317, 319, 321, 323, 325, and 327 (see Fig. 17). The deformations are shown in Figs. 27 and 31. The amount of these deformations at points across the

slab for loads of 200 lb. and 300 lb. per sq. ft. is shown in Fig. 35. It is apparent that a somewhat higher stress existed in one beam than in the other. Taking this into consideration, the compressive stress varies quite uniformly from one beam to the other, and the full width of the floor slab may be said to be effective in taking compression. The overhang (counting to the midpoint between beams) is 6½ times the thickness of slab. It will be noticed that the conclusions are the same as given for the Wenalden building test.

Readings were also taken on the under side of the floor slabs parallel to the beams at three places (No. 1205, 1211, and 1213), but the conditions attending the location of these points do not permit conclusions to be drawn.

Floor Slab.—Measurements were taken on the floor slab in

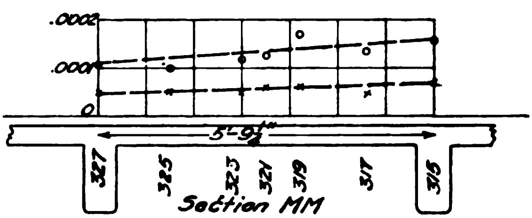


FIG. 35.—DIAGRAM SHOWING DISTRIBUTION OF COMPRESSIVE DEFORMATION ACROSS FLANGE OF TEE BEAMS.

the direction of its span at three places on the under side and at one place on the upper side immediately above one of the lower measurements. These gauge lines were No. 277 on the under side of the slab close to a girder (Fig. 17), No. 279 on the under side of the slab 5 ft. from the edge of the girder, No. 309 (Fig. 18) on the upper surface immediately above No. 279, and No. 1203 (Fig. 17) on the under side half way between girders. The measurements are plotted in Figs. 30 and 31. As might be expected from being close to the girder and near the level of its neutral axis, No. 277 showed little deformation. The pair of gauge lines (No. 279 and 309) shows less deformation than would be calculated by the ordinary beam formula, but perhaps not less than would be the case if the tensile strength of the concrete is considered to be quite effective. The reading of No. 1203 was even smaller than 279.

All the stresses found in the floor slab were low. The deformations parallel to the beams were discussed under T-beams.

Bond Stresses.—At the ends of the beams the reinforcing bars lapped over the center line of the girder a distance of 15 in. An effort was made to determine whether there was a movement of one of these bars with reference to the adjoining concrete and with reference to the adjoining bar; also whether the deformation in the stub end of the reinforcing bar was the same as in the adjoining bar. Fig. 36 shows the location of the reinforcing bars with reference to each other, and the position of the gauge lines. No. 312–14 in comparison with No. 312 and 314 will indicate any relative movement of one bar with respect to the other, and No. 312c and 314c in comparison with No. 312 and 314, respectively will indicate any movement of the bars with respect to the concrete.

It appears possible that the initial reading of No. 314 is

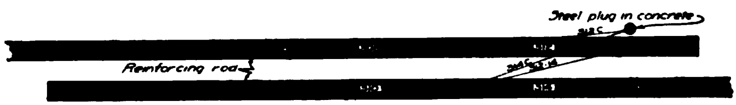


FIG. 36.—ARRANGEMENT OF GAUGE POINTS TO TEST FOR MOVEMENT OF BAR RELATIVE TO CONCRETE.

slightly in error, and the remarks already made about quantitative interpretation of results and the chances for variations in stresses in adjacent bars or in adjoining concrete should be borne in mind in studying the results. It seems evident that No. 314 (on the lapped bar) records considerable less stress than No. 312. The measurements indicate a possibility that the right-hand point of gauge line No. 314 has moved to the right relatively to the right-hand point of No. 312, though this amount may not be more than the amount of initial slip necessary to develop the requisite bond stress. The measurements taken have no bearing on whether the left-hand point of No. 314 has moved. The measurements also indicate that there was no motion of the left-hand point on the reinforcing bar (No. 312 gauge line) relatively to the concrete at its side, though it must be borne in mind that the point taken was so close to the bar that only slip and not distortion of concrete could be measured.

Web Deformations.—No diagonal tension cracks were visible on any of the beams or girders.

In girder 4 measurements were taken on the diagonal portion of a reinforcing bar, one of the bars which is provided to take negative bending moment. This is shown in Fig. 17, Section K-K. The gauge lines are No. 222, 224, and 226 The position of the gauge lines is also shown in Fig. 16. The measurements are plotted in Fig. 32. It was impracticable to measure the deformation at a point closer to the support. The measurements show about the same stress at No. 222 and 224, perhaps 5,000 lb. per sq. in. The stress at No. 226 is materially less. It is not improbable that there was tension in this rod throughout its length. As there was considerable compression measured in the gauge lines on the bottom of the girder below No. 222, it seems probable that a crack was formed in the top of the floor slab somewhere above No. 222, but as this space was filled in with bags of cement no observation was made during the test, and inspection of this space after the load was removed seems to have been overlooked. At the other end of the girder, near column 6, a fine test crack was found on the upper surface of the floor 2 in. from the face of the column extending across the width of the girder and beyond. This extended through the floor. A similar crack was observed on girder 3 near column 15.

Gauge line No. 228 is on a stirrup (see Fig. 16). This stirrup is in an inclined position. It is not known what bar it is intended to be connected with, nor whether there is connection with a tension bar. The gauge line is in a region of the beam where horizontal compressive stresses may be expected. The measurement in the stirrup at the first increment of load shows tension (see Fig. 32) and subsequent increments give compression. It should be noted that readings could not be taken on the upper end of the stirrup. If the upper ends are merely bent out into the floor slab it is hard to see that the stirrup may be expected to be useful in transmitting web stresses.

In beam 9 (see Fig. 17, Section L-L, gauge line No. 218) measurement was taken on the diagonal portion of a reinforcing bar which is carried through the girder at its top and a few inches beyond. See also Fig. 16. This shows a tension of 3,000 to 5,000 lb. per sq. in. (See Fig. 32.) This bar was inaccessible from the

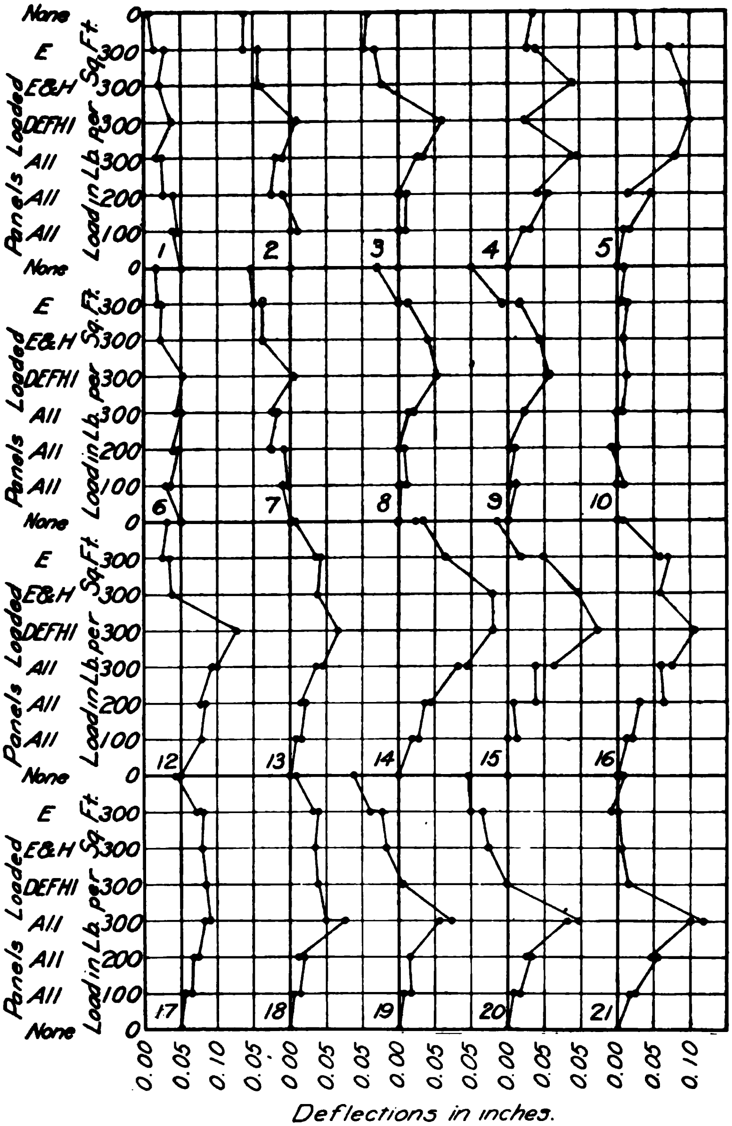


FIG.—37. LOAD DEFLECTION DIAGRAMS.

top of the floor, but the gauge lines on the companion bar (No. 324 and 318) show about 5,000 and 9,000 lb. per sq. in. Measurements in the diagonal portion of a single-bend bar (gauge lines No. 216 and 214, Fig. 17) which extends only to the center of the supporting girder indicate a small compression in the bar (see Fig. 32). A stirrup, which like the one in the girder was close to the end of the beam and was inclined so that its lower end was nearer the support than its upper, showed shortening of the stirrup (see gauge line No. 212, Figs. 16, 17, and 32). In both cases,

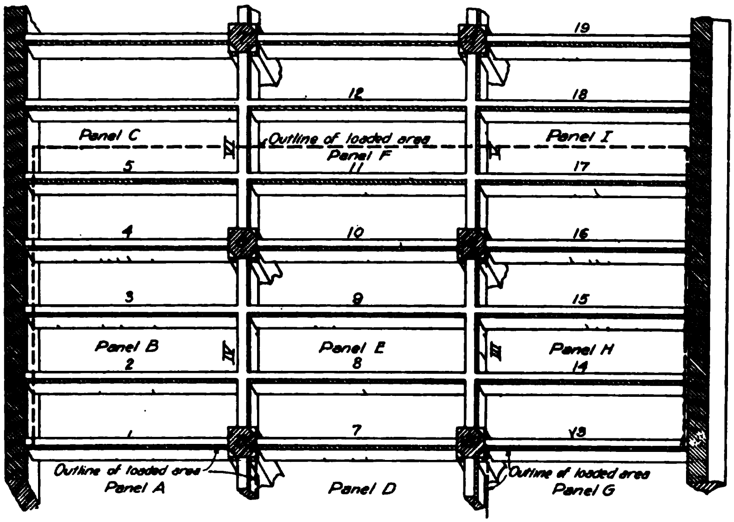


FIG. 38.—CABINET PROJECTION SHOWING BEAMS AND GIRDERS AND POSITION OF TEST CRACKS.

the arrangement was such that the stirrup could hardly be effective.

The amount of the vertical shear in the beams and girders was such that diagonal tension cracks might be expected except for the small tensile stresses in the top of the girder and the end constraint which seems to have been developed in both beams and girders.

Deflections.—The deflections of the beams (including that due to deflection of girder) and the deflections of girders are given in Fig. 37. The location of the deflection points is shown in Fig. 19.

The effect of time upon the deflection is shown by the increase in deflection under constant load. The change when portions of the load had been removed may be due in part to the time element and in part to the effect of location of the load on the panels. The deflections seem relatively small, especially when compared with deflections obtained in laboratory tests of beams carrying the same loads. The effect of the time element is indicated on these diagrams. The conditions were such that the supports were subject to possible displacement by workmen.

Columns.—Readings were taken on the four faces of Column No. 5 just below the girders, but the results are not consistent enough to warrant attempting drawing conclusions.

Test Cracks.—Fine tension cracks were observed in the lower part of the beams and girders. The location of the observed cracks is shown on Fig. 38. The appearance of these fine cracks is similar to those observed in laboratory tests. They would not be noticed without specially careful examination.

The floor cracks already mentioned indicate the development of the tensile stresses in the beams and girders at the support.

The limitation of space and time have prevented the presentation of other matters which were observed in the tests. For example, the observations on deformations during the 48 hours time with the full loading showed in general a slight increase in the deformations in the reinforcement and in the concrete. It is hoped to take up some of these matters at another time. It was not possible to give full attention to every feature upon which information was sought, and in some cases isolated points were used with a view of determining tendencies, and in these naturally there is less certainty in the indications.

PART II. TEST OF A NEW TYPE OF FLAT SLAB FLOOR CONSTRUCTION.

Building.—The Powers Building, Fig. 39, is a three story and basement warehouse located at Minneapolis, Minn. The exterior walls of the building are bearing walls and there is one row of columns through the middle of the building. The floors are flat slabs, reinforced in two directions with high elastic limit deformed bars. Fig. 40 shows a basement and first floor plan

of the building, Fig. 41 the reinforcement in the floor tested and Fig. 42 the reinforcement in place.

General Outline of Test.—The first floor of the building was selected for the test by the Building Inspector of the City of Minneapolis, as this represented an acceptance test for the new

FIG. 39.—POWERS BUILDING, MINNEAPOLIS, MINN.

design. At the time of the test the floor was about 3 months old. Four panels were loaded to 200 lb. per sq. ft. (the design load) and then two panels were loaded to twice this amount. Cement in bags was used for the load, piled in piers to prevent arching. The load was arranged as shown in Fig. 44 until the design load was reached, while for the maximum load Panels 2 and 3 only

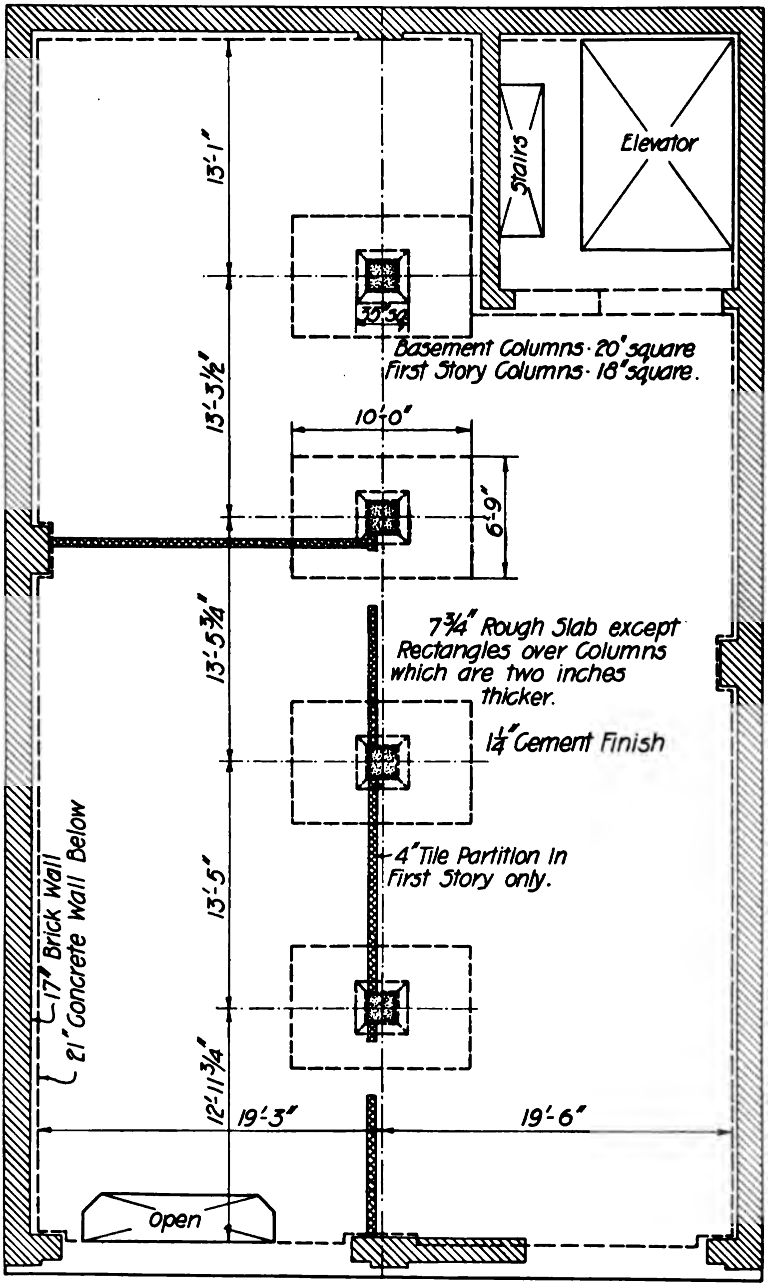


FIG. 40.—PLAN OF POWERS BUILDING, BASEMENT AND FIRST FLOOR.

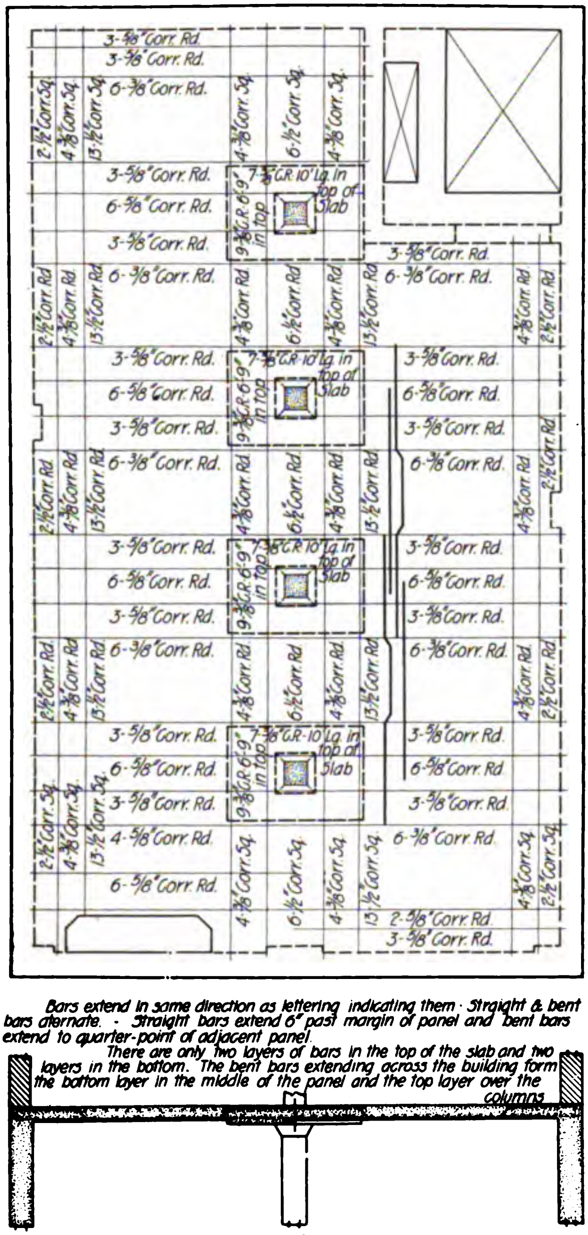


FIG. 41.—DIAGRAM SHOWING ARRANGEMENT OF REINFORCEMENT.

were loaded. The wide aisles shown were necessary for the accommodation of the instruments and observers. All tabulated loads per square foot and the loads used in plotting are in every case the total load on the panel divided by the area of the panel; the intensity of the load under the piers, of course, is greater. Deflections of the various panels were measured under different stages of loading and also the deformation of the reinforcement and concrete due to these loads.

FIG. 42.—REINFORCEMENT IN PLACE, POWERS BUILDING.

Instruments.—The deflections were measured with a deflectometer, see Fig. 1.

Deformations in the reinforcement and concrete were measured with an 8-in. Berry strain gauge, Fig. 43, which reads direct to 1/2000 of an inch, or by estimation to one-fourth of this amount. As slight variation is possible, 5 readings were taken at each point and the average of these assumed to be correct. Readings were taken at intervals throughout the test on standard bars and on standard points placed in unstressed portions of the concrete; the temperature corrections so observed have been

made to the readings. There are many difficulties involved in measuring deformations under conditions such as exist in a test of this kind. In view of these difficulties extreme accuracy can-

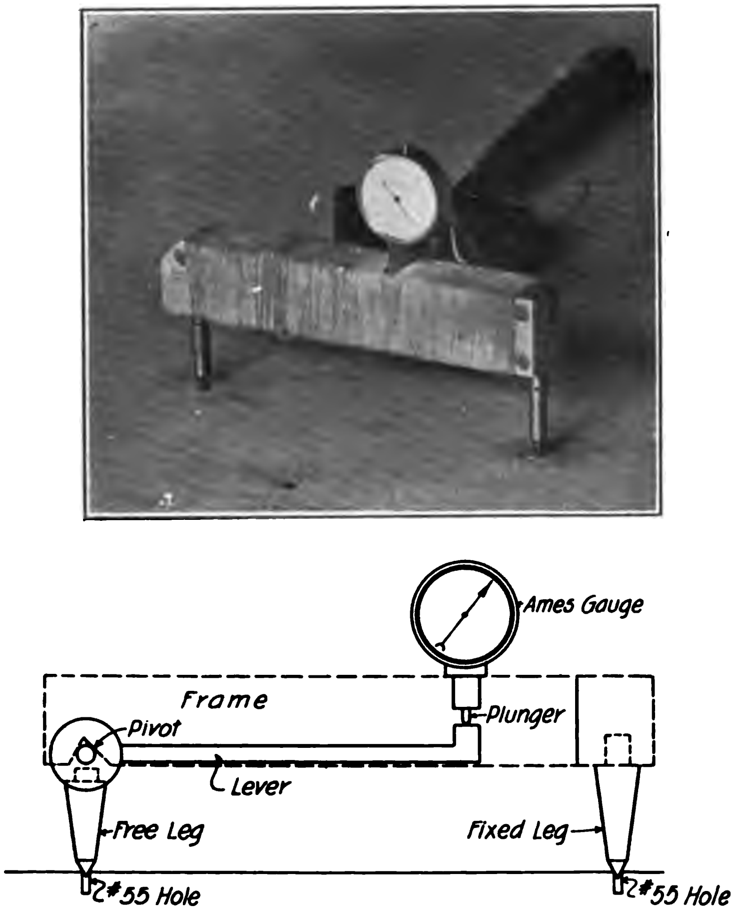


FIG. 43.—BERRY STRAIN GAUGE.

not be hoped for, but serviceable results can be obtained by a trained observer. Observations were made by W. A. Slater, University of Illinois, and F. J. Trelease of the Research Department, Corrugated Bar Company.

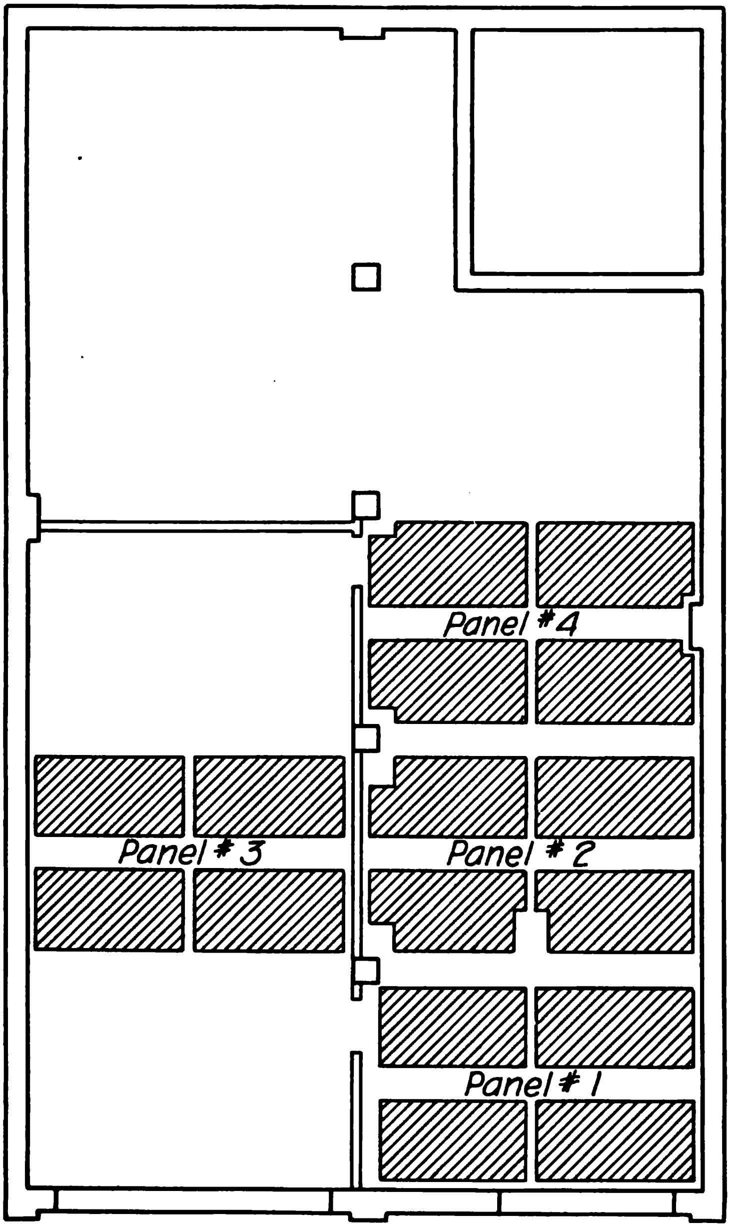


FIG. 44.—ARRANGEMENT OF LOADING.

Application of Load.—The test continued for 9 days, July 27 to August 4, inclusive, 1911. The loading was in 10 stages as

TABLE VII.—SCHEDULE OF LOADING OPERATIONS.

Stage.	Load in Place		Load in Pounds per Panel and per Square Foot.							
	From	To	Panel 1.		Panel 2.		Panel 3.		Panel 4.	
1	1.45 P.M., 7-27	8.40 A.M., 7-28	13,300	52	13,300	51	13,300	51	13,300	51
2	11.10 A.M., 7-27	2.00 P.M., 7-28	33,440	132	33,440	128	33,440	129	33,440	128
3	4.45 P.M., 7-28	8.45 A.M., 7-29	53,200	210	53,200	204	53,200	206	53,200	204
4	10.45 A.M., 7-29	1.25 P.M., 7-29	34,200	135	72,200	276	72,200	280	34,200	131
5	3.40 P.M., 7-29	5.40 P.M., 7-29	15,200	60	91,200	350	91,200	354	15,200	58
6	7.00 P.M., 7-29	11.00 A.M., 7-31	....	....	106,400	408	106,400	414	....	....
7	3.15 P.M., 8-1	4.25 P.M., 8-1	....	....	96,900	370	50,920	198	....	....
8	8.20 A.M., 8-2	9.15 A.M., 8-2	....	....	106,780	408	....	....	....	....
9	4.30 P.M., 8-2	7.15 A.M., 8-3	....	....	53,010	202	....	....	....	....
10	10.35 A.M., 8-3	3.45 P.M., 8-3	....	....	21,660	83	....	....	....	....

NOTE.—Started placing Stage 1 at 10 A. M., 7-27-1911. All load removed at 4.20 P. M., 8-3-1911.

will be noted from Table VII, which shows the arrangement of the load at various stages and the length of time each load remained on

FIG. 45.—DESIGN LOAD IN PLACE ON FOUR PANELS.

the floor. Readings were first taken on all points with the floor unloaded and then a load approximately equivalent to 50 lb.

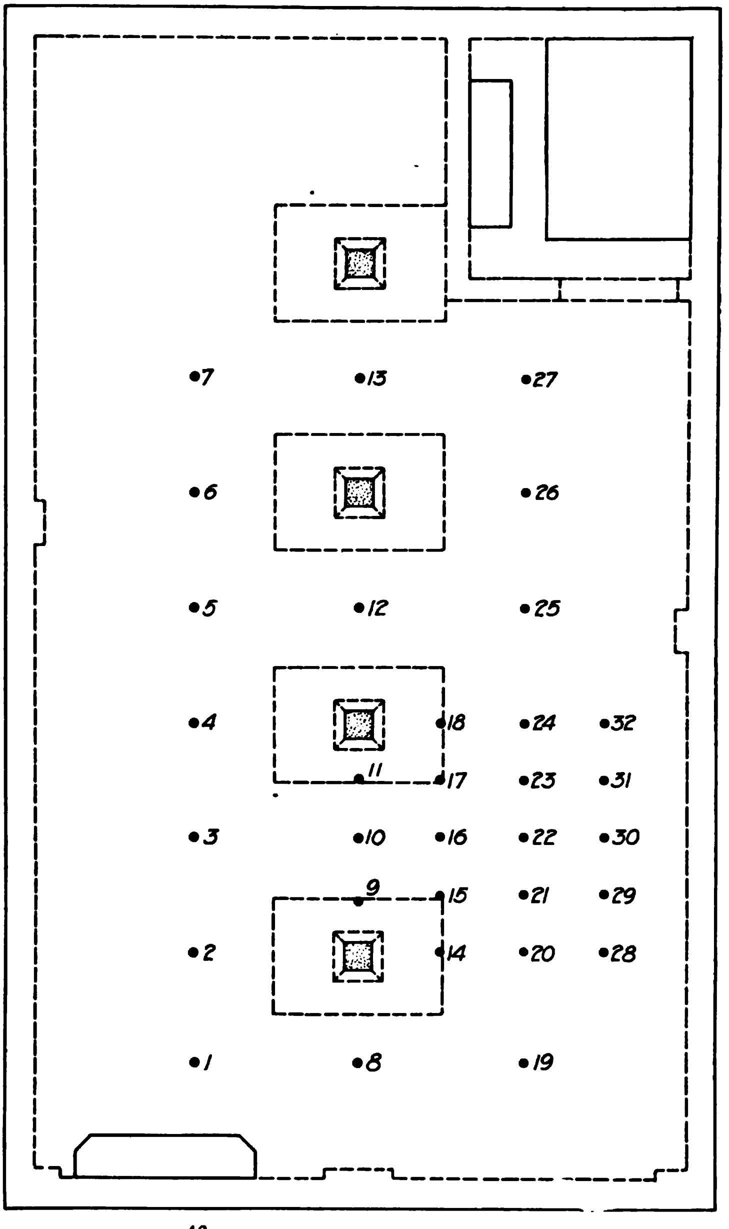


FIG. 46.—LOCATION OF DEFLECTION POINTS.

per sq. ft. was applied over 4 panels, Fig. 44. Another set of readings was taken and the load increased to 125 lb. per sq. ft. This method of alternate loading and reading obtained throughout the test. When the design load of 200 lb. was reached, however, the load on Panels 1 and 4 was moved by stages to Panels 2 and 3, so that finally Panels 1 and 4 were completely unloaded and Panels 2 and 3 were loaded to about 400 lb. per sq. ft. The

TABLE VIII.—DEFLECTIONS IN INCHES.

Set	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Load	DL	1	1	2	3	3	4	5	6	6	6	7	9	9	10	DL
Gauge Point																
1	0	-.005	.000	.004	.011	.016	.026	.043	.065	.082	.079	.059	.040	.040	.042	.....
2	0	.005	.015	.032	.052	.060	.091	.143	.197	.237	.....	.174	.112	.110	.114	.....
3	0	.014	.026	.057	.079	.090	.139	.217	.294	.351	.344	.271	.159	.159	.161	.155
4	0	.004	.034	.055	.066	.073	.102	.144	.195	.243	.....	.196	.102	.108	.113	.....
5	0	.000	.005	.003	.008	.013	.017	.035	.050	.063	.070	.070	.047	.050	.048	.....
6	0	-.002	.010	.012	.010	.008	.010	.015	.020	.028	.....	.026	.012	.015	.016	.....
7	0	-.001	.007	.008	.003	.003	.005	.006	.009	.014	.....	.....	.010	.....	.014	.....
8	0	-.016	-.003	.000	.009	.012	.007	.002	-.002	.000	.004	-.002	-.003	-.005	.003	.....
9	0	.004	.012	.017	.019	.021	.028	.037	.046	.055	.....	.....	.031	.030	.026	.....
10	0	.038	.043	.039	.046	.056	.067	.076	.101	.112	.120	.104	.070	.066	.059	.056
11	0	.000	.000	.006	.014	.014	.020	.033	.035	.049	.....	.....	.023	.030	.026	.....
12	0	.001	.002	.005	.009	.009	.011	.009	.008	.009	.010	.008	.006	.007	.007	.009
13	0	-.001	.012	.015	.013	.013	.013	.015	.015	.022	.....	.....	.011	.....	.015	.....
14	0	.008	.022	.051	.111	.126	.134	.153	.152	.135	.....	.....	.120	.115	.095	.....
15	0	.016	.019	.054	.126	.146	.163	.182	.209	.216	.....	.....	.194	.187	.156	.....
16	0	.017	.028	.067	.142	.155	.175	.220	.242	.240	.....	.....	.210	.207	.171	.131
17	0	.012	.023	.060	.133	.149	.163	.190	.198	.205	.....	.....	.177	.169	.141	.....
18	0	.010	.019	.049	.118	.138	.141	.144	.137	.147	.....	.....	.136	.130	.110	.....
19	0	.013	.028	.068	.137	.158	.153	.145	.136	.132	.148	.151	.130	.131	.120	.232
20	0	.022	.035	.085	.205	.238	.251	.273	.283	.307	.....	.320	.272	.260	.226	.....
21	0	.021	.033	.081	.213	.255	.278	.320	.348	.378	.....	.....	.327	.308	.265	.....
22	0	.020	.037	.079	.224	.264	.294	.346	.374	.413	.421	.424	.350	.333	.284	.214
23	0	.028	.038	.090	.236	.278	.303	.326	.363	.401	.....	.....	.350	.312	.286	.....
24	0	.019	.041	.106	.249	.286	.289	.318	.316	.349	.....	.363	.313	.304	.259	.....
25	0	.016	.027	.069	.154	.156	.179	.166	.148	.148	.149	.156	.149	.151	.130	.109
26	0	.008	.021	.058	.321	.341	.326	.306	.....	.137	.....	.....	.....	.....	.....	.....
27	0	-.001	.006	.010	.021	.029	.034	.027	.023	.027	.....	.....	.022	.....	.021	.....
28	0	.014	.023	.061	.158	.184	.196	.206	.229	.243	.....	.....	.214	.200	.179	.....
29	0	.013	.022	.169	.176	.211	.230	.266	.291	.319	.....	.....	.287	.268	.242	.....
30	0	.015	.018	.065	.179	.214	.234	.273	.305	.341	.....	.....	.299	.284	.254	.....
31	0	.017	.029	.079	.196	.228	.248	.285	.309	.342	.....	.....	.302	.290	.255	.....
32	0	.018	.031	.078	.196	.229	.236	.256	.268	.287	.....	.....	.270	.261	.230	.....

load was next removed from Panel 3 and finally from Panel 2, readings being taken at intervals during the unloading. Fig. 45 shows the design load in place on the four panels.

Data.—Deflection readings were taken at 32 points, Fig. 46. The deflections are given in Table VIII and are plotted in Figs. 47 and 48. The load plotted in these figures and in the stress curves is the load per square foot on the central panels. The stage of loading can be obtained by referring to Table VIII.

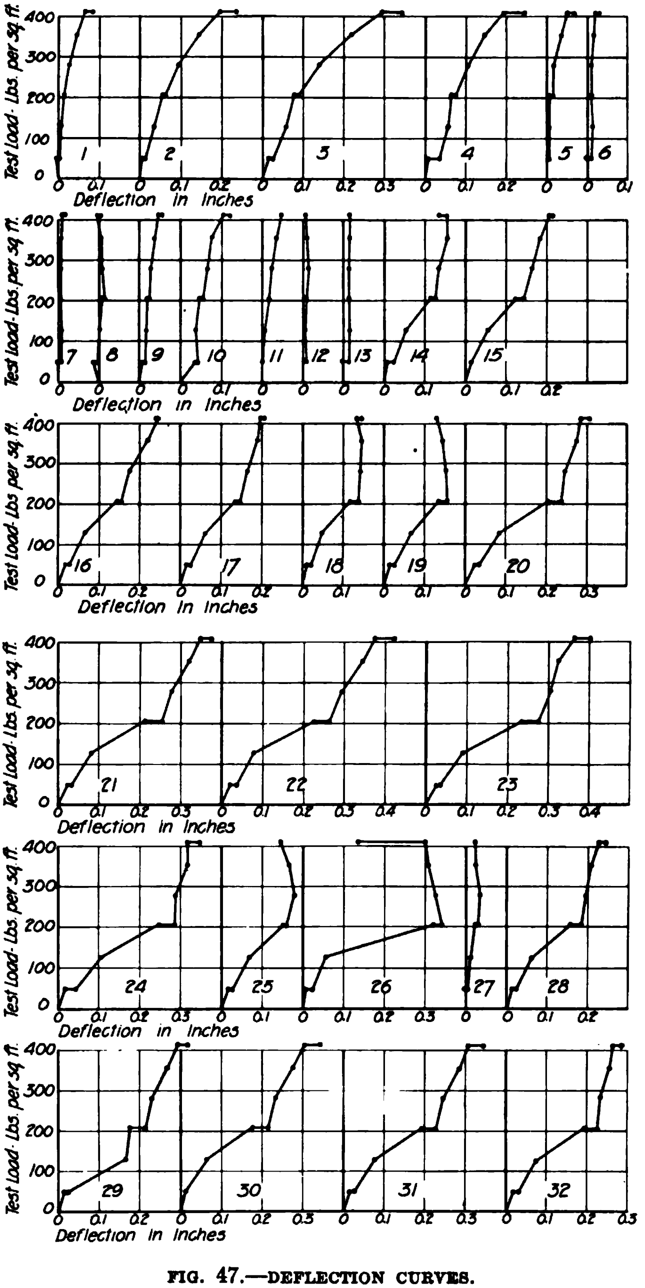


FIG. 47.—DEFLECTION CURVES.

Deformation of the reinforcement was read at 30 points and of the concrete at 31 points. The location of these points is shown in Figs. 49 and 50. Table IX gives the embedment to center of bar; thickness of the slab and finish coat; net thickness

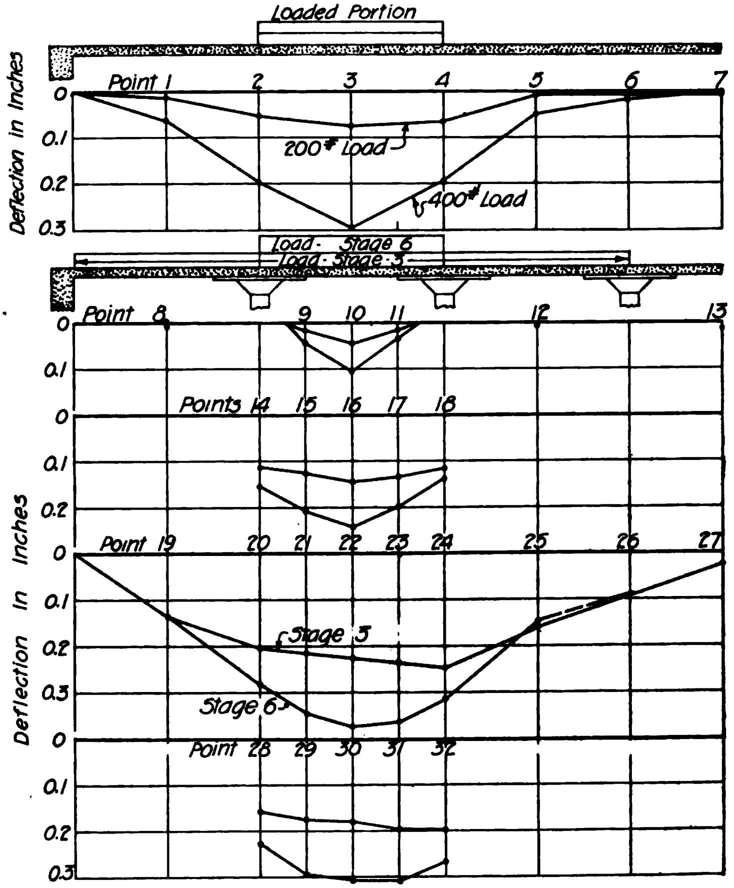
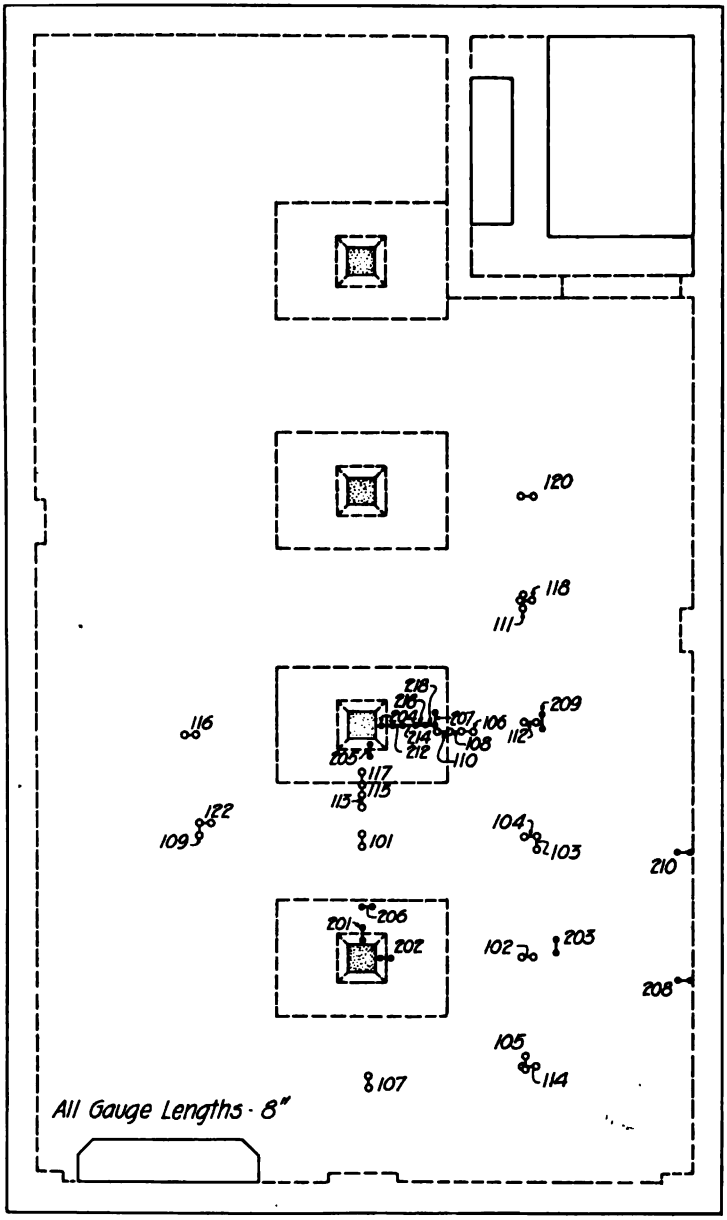


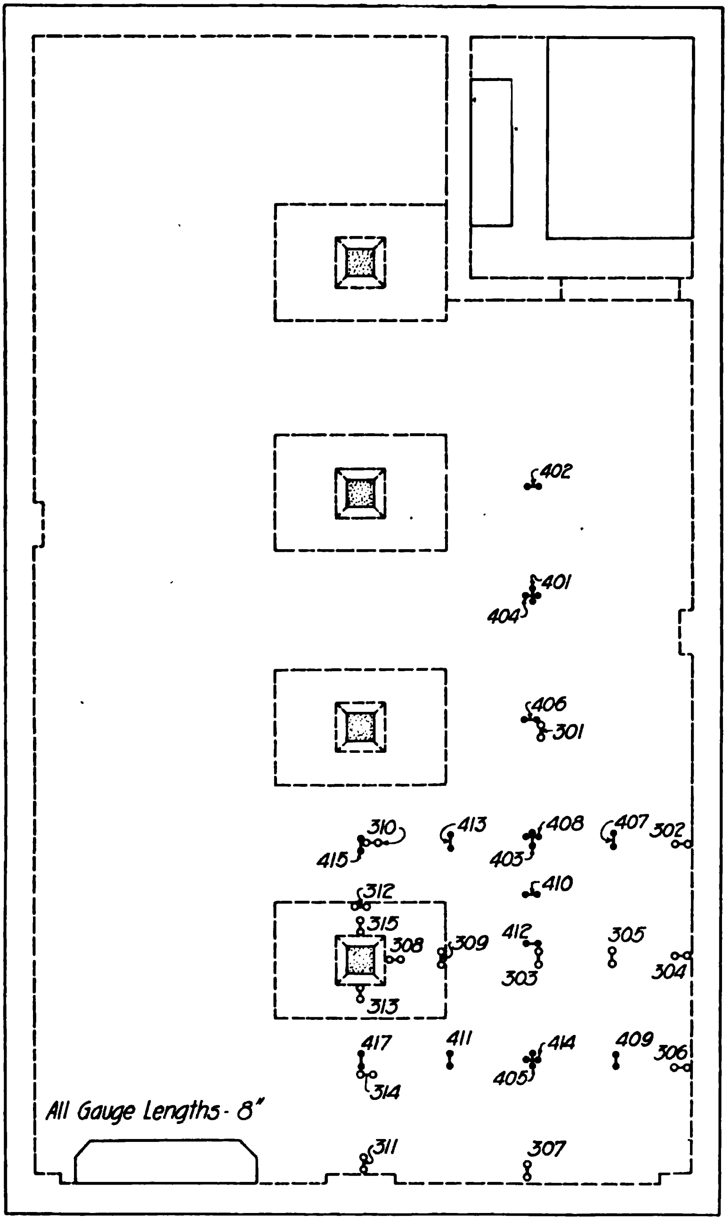
FIG. 48.—SECTIONS OF FLOOR SHOWING DEFLECTIONS.

of the rough concrete slab, deducting the thickness of the finish coat from the total thickness of slab, as the finish coat was found to be loose in many places and did not have much load-carrying capacity; and also the effective depth of the reinforcement on which the strain was measured.



●—● Indicates Points on Top of Slab
○—○ Indicates Points on Bottom of Slab

FIG. 49.—LOCATION OF DEFORMATION GAUGE POINTS OR REINFORCEMENT.



●—● Indicates Points on Top of Slab
○—○ Indicates Points on Bottom of Slab.

FIG. 50.—LOCATION OF DEFORMATION GAUGE POINTS ON CONCRETE.

The observed elongations and corresponding unit stresses, based on an assumed modulus of 30,000,000, in the reinforcement are given in Table X. Unit reinforcement stresses are plotted against the load per square foot on Panel 2 in Figs. 51 and 52. A series of readings taken by Mr. Slater on one of the bars over the column cap shows that a bond stress of 135 lb. per sq. in. under maximum loading existed at that point.

TABLE IX.—DIMENSIONS AT DEFORMATION GAUGE POINTS.

Gauge Point.	Embedment of Bar to center, in.	Thickness of		Net Thickness of Slab, in in.		Effective Depth of Bars, in.	
		Slab, in.	Finish Coat, in.	Actual.	From Plans.	Actual.	From Plans.
101	1 3/4	9 1/8	1 1/4	7 7/8	7 3/4	7 3/8	7 5/8*
102	1	9 1/8	"	7 7/8	"	8 1/8	8 1/4*
103	1 11/16	9 1/8	"	7 7/8	"	7 7/16	7 5/8*
104	1 5/16	9 1/8	"	7 7/8	"	7 15/16	8 1/4*
105	1 1/2	8 7/8	"	7 5/8	"	7 3/8	7 5/8*
106	1 1/4	....	....	....	....	....	....
107	1 11/16	9 5/16	"	8 1/16	"	7 5/8	7 5/8*
108	1 1/8	....	....	....	....	....	....
109	2 3/16	8 3/8	"	7 1/8	"	6 3/16	7 5/8*
111	1 13/16	8 15/16	"	7 11/16	"	7 1/8	7 5/8*
112	1 7/16	9	"	7 3/4	"	7 9/16	8 1/4*
113	1 11/16	....	....	....	....	....	....
114	0 15/16	8 7/8	"	7 5/8	"	7 15/16	8 1/4*
115	1 15/16	....	....	....	....	....	....
116	1 1/8	....	....	....	....	....	....
118	1 1/4	8 15/16	"	7 11/16	"	7 11/16	8 1/4*
120	0 7/16	....	....	....	....	....	....
122	1 1/8	8 3/8	"	7 1/8	....	7 1/4	8 1/4*
201	3 7/16	10 5/8	1 1/4	9 3/8	9 3/4	7 3/16	8 3/8
202	3 3/8	10 5/8	1 1/4	9 3/8	9 3/4	7 1/4	9
203	2 1/8	9 1/8	1 1/4	7 7/8	7 3/4	7	6 3/8
204	2 7/16	11 1/16	1 1/4	9 15/16	9 3/4	8 5/8	9
205	3 3/8	10 7/8	1 1/4	9 5/8	9 3/4	7 1/2	8 3/8
206	3 1/16	10 15/16	1 1/2	9 7/16	9 3/4	7 7/8	9
207	2 7/8	11 1/4	1	10 1/4	9 3/4	8 3/8/	8 3/8
208	2 5/16	8 13/16	1 3/16	7 5/8	7 3/4	6 1/2	7
209	3 1/4	9	1 1/4	7 3/4	7 3/4	5 3/4	6 3/8
210	2 7/16	9 1/16	1 1/16	8	7 3/4	6 5/8	7

* Assuming 1½ in. finish coat.

Table XI gives the observed elongations and corresponding unit concrete stresses. These concrete stresses are based on an assumed modulus of 2,000,000 and are plotted against loads per square foot in Figs. 53 and 54.

A summary of the unit stresses is given in Table XII, the stresses given being the maximum probable values.

During the test, cracks were carefully searched for with an electric light and recorded as found. Their location is shown in Fig. 55. Most of these cracks were very minute and difficult

TABLE X.—DEFORMATIONS IN INCHES AND REINFORCEMENT STRESSES IN LB. PER SQ. IN.

Set	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Load	DL	DL	1	1	2	3	3	4	5	6	6	7	8	9	10	DL
Gauge Point																
101	0	0.9	0.3	-0.1	-0.1	0.2	.....	0.7	1.1	1.6	2.0	2.2	2.4	1.9	1.6	2.8
	0	1685	560	-190	-190	380	.....	1310	2060	3000	3750	4120	4500	3560	3000	5250
102	0	0.3	-0.3	0.6	0.6	3.3	.....	4.2	4.3	4.3	4.2	4.4	5.6	4.0	3.0	2.7
	0	560	-560	1120	1120	6200	.....	7880	8060	8060	7880	8250	10500	7500	5620	5060
103	0	-0.3	-0.6	-0.9	-0.4	-0.1	.....	0.9	2.7	2.7	3.5	4.2	4.8	3.9	3.4	2.0
	0	-560	-1120	-1690	-750	-190	.....	1690	5060	5060	6560	7880	9000	7310	6380	3750
104	0	-0.5	-0.7	-0.4	0.3	3.6	.....	4.7	5.0	5.7	6.0	7.0	7.7	5.7	4.9	3.7
	0	-940	-1310	.750	560	6740	.....	8810	9380	10690	11250	13120	14440	10690	9190	6940
105	0	0.4	-0.8	.....	-0.4	-0.5	.....	.....	.....	.....	.....	.....	.....	-1.0	-0.3	-1.2
	0	750	-1500	.....	-750	-940	.....	.....	.....	.....	.....	.....	.....	-1880	-560	-2250
106	0	.....	-0.5	.....	0.1	0.2	.....	1.5	8.0	0.6	0.8	.....	.....	.....	.....	.....
	0	.....	-940	.....	190	380	.....	2810	15000	1120	1500	.....	.....	.....	.....	.....
107	0	*0.7	-0.2	.....	-0.5	0.0	.....	.....	.....	.....	.....	.....	.....	0.0	0.0	-0.5
	0	1310	-380	.....	-940	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	-940
108	0	.....	0.3	.....	0.1	0.3	.....	0.3	-0.8	-3.1	0.3	.....	.....	.....	.....	.....
	0	.....	560	.....	190	190	.....	560	-1500	-5810	560	.....	.....	.....	.....	.....
109	0	0.8	-0.1	.....	-0.5	0.2	.....	0.9	1.8	2.4	.20	2.2	.....	0.8	0.8	1.2
	0	1500	-190	.....	-940	380	.....	1690	3380	4500	3750	9120	.....	1500	1500	2250
110	0	.....	-0.3	.....	-1.6	-1.8	.....	-2.1	-2.4	-3.0	-2.5	.....	.....	.....	.....	.....
	0	.....	-560	.....	-3000	-3380	.....	-3940	-4500	-5600	-4600	.....	.....	.....	.....	.....
111	0	.....	-0.3	.....	-0.4	0.2	.....	.....	.....	.....	.....	.....	.....	-0.3	.....	-1.6
	0	.....	-560	.....	-750	380	.....	.....	.....	.....	.....	.....	.....	-560	.....	-3000
112	0	-0.5	0.1	0.5	0.9	5.3	.....	6.3	5.4	6.0	6.1	.....	7.6	5.9	5.6	4.0
	0	-940	190	940	1690	9940	.....	11810	10120	11250	11440	.....	14250	11060	10500	7500
113	0	.....	-0.6	.....	-1.0	-0.6	.....	0.4	1.2	0.3	1.4	.....	.....	.....	.....	.....
	0	.....	-1120	.....	-1880	-1120	.....	750	2250	560	2620	.....	.....	.....	.....	.....
114	0	0.3	0.2	.....	0.3	1.2	.....	2.1	2.7	1.5	.....	.....	.....	2.0	1.2	1.7
	0	560	380	.....	560	2250	.....	3940	5060	2810	.....	.....	.....	3750	2250	3190
115	0	.....	-0.5	.....	-0.1	-0.8	.....	0.3	0.9	0.1	-0.1	.....	.....	.....	.....	.....
	0	.....	-940	.....	-190	-1500	.....	560	1690	190	190	.....	.....	.....	.....	.....
116	0	1.2	-0.5	.....	-0.2	0.2	.....	0.7	1.3	1.5	1.7	.....	.....	1.2	1.2	0.8
	0	2250	-940	.....	-380	380	.....	1310	2440	2810	3190	.....	.....	2250	2250	1500
117	0	.....	-0.9	.....	-0.9	-0.6	.....	-0.7	-1.4	-1.5	-0.5	.....	.....	.....	.....	.....
	0	.....	-1690	.....	-1690	-1120	.....	-1310	-2620	-2810	-940	.....	.....	.....	.....	.....
118	0	.....	-0.3	.....	0.5	2.8	.....	2.9	1.9	2.4	2.1	.....	.....	2.7	....	1.3
	0	.....	-560	.....	940	5250	.....	5440	3560	4500	3940	.....	.....	5060	.....	2440
120	0	.....	-0.2	.....	0.7	1.7	.....	.....	.....	.....	.....	.....	.....	.....	.....	0.5
	0	.....	-380	.....	1310	3190	.....	.....	.....	.....	.....	.....	.....	.....	.....	940
122	0	0.8	-0.3	.....	0.2	1.5	.....	1.4	3.0	3.9	4.0	3.4	.....	1.8	2.1	2.4
	0	1500	-560	.....	380	2810	.....	2620	5620	7310	7500	6380	.....	3380	3940	4500
201	0	0.8	1.0	0.8	1.2	1.6	1.5	2.0	1.8	3.1	3.7	3.1	.....	2.8	2.4	2.5
	0	1500	1880	1500	2250	3000	2810	3750	3380	5810	6940	5810	.....	5250	4500	4690
202	0	0.5	1.1	0.4	2.7	7.0	7.6	8.1	9.2	9.1	10.1	9.1	9.5	7.8	5.8	5.2
	0	940	2060	750	5060	13120	14250	15190	17250	17060	18940	17060	17810	14620	10880	9750
203	0	0.4	0.5	-0.4	-0.1	0.5	-0.1	0.1	-0.8	-0.5	-0.6	.....	.....	.....	.....	-0.2
	0	750	940	-750	-190	940	-190	190	-1500	-940	-1120	.....	.....	.....	.....	-380

TABLE X.—*Continued.*

Set	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Load	DL	DL	1	1	2	3	3	4	5	6	6	7	8	9	10	DL
Gauge Point																
204	0	0.4	-0.1	0.6	2.9	9.4	9.7	10.4	11.5	11.7	12.3	12.0	12.2	9.7	7.1	5.80
	0	750	-190	1120	5440	17620	18190	19500	21560	21940	23060	22500	22880	18190	13310	9380
205	0	0.1	0.3	0.6	0.9	1.6	1.7	.....	3.3	4.7	5.2	4.0	.....	2.3	2.3	1.3
	0	190	560	1120	1690	3000	3190	.....	6190	8810	9750	7500	.....	4310	4310	2440
206	0	0.4	.....	0.7	3.1	7.7	8.8	10.4	12.2	12.4	14.4	.....	.....	10.6	3.2	8.0
	0	750	.....	1310	5810	14440	16500	19500	22880	23250	27000	.....	.....	19880	17440	15000
207	0	-0.8	-0.5	.....	0.1	0.6	1.6	0.7	0.5	0.7	.....	.....	.....	.....	.....	-0.8
	0	-1500	-940	.....	190	1120	3000	1310	940	1310	.....	.....	.....	.....	.....	-1500
208	0	-0.2	0.7	0.8	3.2	4.6	4.9	6.4	5.6	6.7	7.0	6.3	....	4.8	3.2	1.4
	0	-380	1310	1500	5620	8620	9190	12000	10500	12560	13120	11810	.....	9000	6000	3190
208	A	.....	.....	.....	.....	.....	0.3	1.0	0.6	1.1	1.9	.....	.....	0.6	0.9	0.4
	.....	.....	.....	.....	.....	.....	560	1880	1120	2060	3560	.....	.....	1120	1690	750
209		-0.1	-0.3	-0.3	-0.3	-0.5	-0.3	.....	.....	.....	.....	.....	.....	.....	.....	-0.8
	0	-190	-560	-560	-560	-940	-560	.....	.....	.....	.....	.....	.....	.....	.....	-1500
210	0	-0.3	0.9	0.1	0.5	2.9	3.1	4.2	4.4	4.5	5.0	.....	.....	3.2	4.2	-2.9
	0	-560	1690	1880	2810	5440	5810	7780	8250	8440	9380	.....	.....	6000	2810	-5440

TABLE XI.—DEFORMATIONS IN INCHES AND CONCRETE STRESSES IN LB. PER SQ. IN.

Set	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Load	DL	DL	1	1	2	3	3	4	5	6	6	7	8	9	10	DL
Gauge Point																
301	0	.....	-0.8	-0.5	-0.2	0.6	.....	0.3	-0.1	.....	.....	.....	.....	.....	.....	-0.2
	0	.....	-100	-60	-25	75	.....	40	-10	.....	.....	.....	.....	.....	.....	-25
302	.....	.....	.....	0.1	-0.2	-0.9	.....	-1.1	-1.2	.....	-1.5	.....	.....	-0.3	-0 2	0.2
	.....	.....	.....	10	-25	-110	.....	-140	-150	.....	-190	.....	.....	-40	-25	25
303	0	-0.3	-0.7	-0.2	-0.2	.....	.....	0.3	0.2	-0.1	-0.1	.....	.....	.....	.....	0.5
	0	-40	-90	-25	-25	.....	.....	40	25	-10	-10	.....	.....	.....	.....	60
304	0	0.8	-0.1	-0.2	-0.7	-2.0	.....	-1.7	-2.6	-2.9	-3.4	-3.1	.....	-2.9	-0.7	-1.5
	0	100	-10	-25	-90	-250	.....	-210	-325	-360	-425	-390	.....	-360	-90	-190
305	0	0.5	-0.7	.....	-0.6	.....	.....	0.3	0.2	0.4	.....	.....	.....	.....	.....	0.1
	0	60	-90	.....	-75	.....	.....	40	25	50	.....	.....	.....	.....	.....	10
306	0	.....	-1.2	.....	-1.6	-1.8	.....	-3.2	-3.9	-.7	.....	.....	.....	.....	.....	-2.3
	0	.....	-150	.....	-200	-225	.....	-400	-360	-340	.....	.....	.....	.....	.....	-290
307	0	.....	0.1	.....	-1.0	-1.2	.....	.....	.....	.....	.....	.....	.....	.....	-0.1	-0.4
	0	.....	10	.....	-125	-150	.....	.....	.....	.....	.....	.....	.....	.....	-10	-50
308	0	0.1	-0.8	-0.7	-2.8	-6.8	.....	-6.8	-8.3	-8.6	-9.4	-8.5	-8.7	-7.1	-6.2	-4.7
	0	10	-100	-90	-350	-850	.....	-850	-1040	-1075	-1175	-1060	-1090	-890	-775	-590
309	0	-0.3	-0.9	-0.7	-0.8	-1.4	.....	-1.8	-1.7	-1.4	-1.7	.....	.....	.....	.....	-0.1
	0	-40	-110	-90	-100	-175	.....	-225	-210	-175	-210	.....	.....	.....	.....	-10
310	0	1.5	-0.3	-0.7	-2.3	-3.9	.....	-5.6	-5.6	-6.3	-7.9	-7.1	.....	-5.1	-4.5	-2.7
	0	190	-40	-90	-290	-490	.....	-700	-700	-790	-990	-890	.....	-640	-560	-340

TABLE XI.—*Continued.*

Set	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Load	DL	DL	1	1	2	3	3	4	5	6	6	7	8	9	10	DL
Point Gauge 311	0	.....	-0.7	.....	-0.5	.....	.....	.....	.....	.....	.....	.....	.....	-0.3	-0.5	-0.6
	0	.....	-90	.....	-60	.....	.....	.....	.....	.....	.....	.....	.....	-40	-60	-75
312	0	.....	-0.4	.....	-0.8	-1.8	.....	-2.9	-3.5	-4.1	-5.5	.....	.....	-2.1	-1.5	-1.0
	0	.....	-50	.....	-100	-225	.....	-360	-440	-510	-690	.....	.....	-260	-190	-125
313	0	-0.4	-0.8	.....	-1.3	-1.6	.....	.....	.....	.....	.....	.....	.....	-1.4	-1.0	-1.2
	0	-50	-100	.....	-160	-200	.....	.....	.....	.....	.....	.....	.....	-175	-125	-150
314	0	0.6	-1.5	.....	-0.4	-0.8	.....	.....	.....	.....	.....	.....	.....	.....	.....	0.1
	0	75	-190	.....	-50	-100	.....	.....	.....	.....	.....	.....	.....	.....	.....	10
315	0	.....	-1.1	-0.1	-1.2	-1.9	.....	-1.3	-3.1	-2.9	-3.6	-3.9	-3.2	-2.4	-3.0	-2.1
	0	.....	-140	-10	-150	-240	.....	-160	-390	-360	-450	-490	-400	-300	-375	-260
401	0	-0.6	-0.6	.....	-0.9	-1.0	.....	-1.0	-0.5	-0.4	.....	.....	.....	.....	.....	-1.1
	0	-75	-75	.....	-110	-125	.....	-125	-60	-50	.....	.....	.....	.....	.....	-140
402	0	-0.4	.....	.....	-0.6	-1.6	.....	.....	.....	.....	.....	.....	.....	.....	.....	-2.4
	0	-50	.....	.....	-75	-200	.....	.....	.....	.....	.....	.....	.....	.....	.....	-300
403	0	-0.4	-0.1	-0.5	-0.5	-1.0	-1.1	-1.4	-2.3	-2.4	-3.4	.....	.....	-2.1	-1.2	-0.8
	0	-50	-10	-60	-60	-125	-140	-175	-290	-300	-425	.....	.....	-260	-150	-100
404	0	-0.1	-1.0	.....	-1.9	-3 2	.....	-4.5	-3.3	-2.6	.....	.....	.....	.....	.....	-3.2
	0	-10	-125	.....	-240	-400	.....	-560	-410	-325	.....	.....	.....	.....	.....	-400
405	0	0.4	0.1	.....	0.2	-0.2	.....	.....	.....	.....	.....	.....	.....	.....	.....	-1.8
	0	50	10	.....	25	-25	.....	.....	.....	.....	.....	.....	.....	.....	.....	-225
406	0	-0.5	-0.5	-1.0	-1.1	-2.6	-3.3	-2.9	-3.1	-2.9	-2.9	.....	.....	-3.2	-2.7	-2.4
	0	-60	-60	-125	-140	-325	-410	-360	-390	-360	-360	.....	.....	-400	-340	-300
407	0	-0.1	-0.2	.....	-0.1	-0.2	-0.5	-0.7	-0.9	-0.8	-1.6	.....	.....	.....	.....	-1.6
	0	-10	-25	.....	-10	-25	-60	-90	-110	-110	-200	.....	.....	.....	.....	-200
408	0	-0.2	-0.8	-0.9	-1.4	-3.0	-3.4	-3.5	-3.8	-3.7	-4.7	-4.7	.....	-3.6	-2.6	-1.9
	0	-25	-100	-110	-175	-375	-425	-440	-475	-460	-590	-590	.....	-450	-325	-240
409	0	-0.2	0.2	.....	.....	-0.9	.....	.....	.....	.....	.....	.....	.....	.....	.....	-1.5
	0	-25	25	.....	.....	-110	.....	.....	.....	.....	.....	.....	.....	.....	.....	-190
410	0	0.5	-0.2	-0.5	-0.3	-2.4	-2.5	-2.7	-3.1	-3.4	-3.6	-4.5	.....	.....	.....	-1.0
	0	60	-25	-60	-40	-300	-310	-340	-390	-425	-450	-560	.....	.....	.....	125
411	0	.....	-0.1	.....	-0.2	-0.3	.....	.....	.....	.....	.....	.....	.....	.....	.....	-1.1
	0	.....	-10	.....	-25	-40	.....	.....	.....	.....	.....	.....	.....	.....	.....	-140
412	0	.....	-0.2	-0.7	-1.3	-3.1	-3.2	-3.3	-3.8	-3.4	-4.0	-4.4	.....	-3.8	-2.4	-1.6
	0	.....	-25	-90	-160	-390	-400	-410	-475	-425	-500	-550	.....	-475	-300	-200
413	0	-0.6	-0.5	-0.6	-0.6	-1.0	-1.5	-2.0	-2.6	-2.7	-3.3	.....	.....	.....	.....	-1.6
	0	-75	-60	-75	-75	-125	-190	-250	-325	-340	-410	.....	.....	.....	.....	-200
414	0	0.5	.....	.....	-0.5	-0.8	.....	-0.8	-1.3	-0.9	-0.8	.....	.....	.....	.....	0.2
	0	60	.....	.....	-60	-100	.....	-100	-160	-110	-100	.....	.....	.....	.....	25
415	0	-0.6	-0.2	-0.7	-0.4	-0.7	-1.4	-1.4	-1.9	-2.4	-3.1	-3.2	.....	-2.6	-1.7	-1.6
	0	-75	-25	-90	-50	-90	-175	-175	-240	-300	-390	-400	.....	-325	-210	-200
417	0	0.1	-0.1	.....	-0.1	-0.2	.....	.....	.....	.....	.....	.....	.....	.....	.....	0.1
	0	10	-10	.....	-10	-25	.....	.....	.....	.....	.....	.....	.....	.....	.....	10

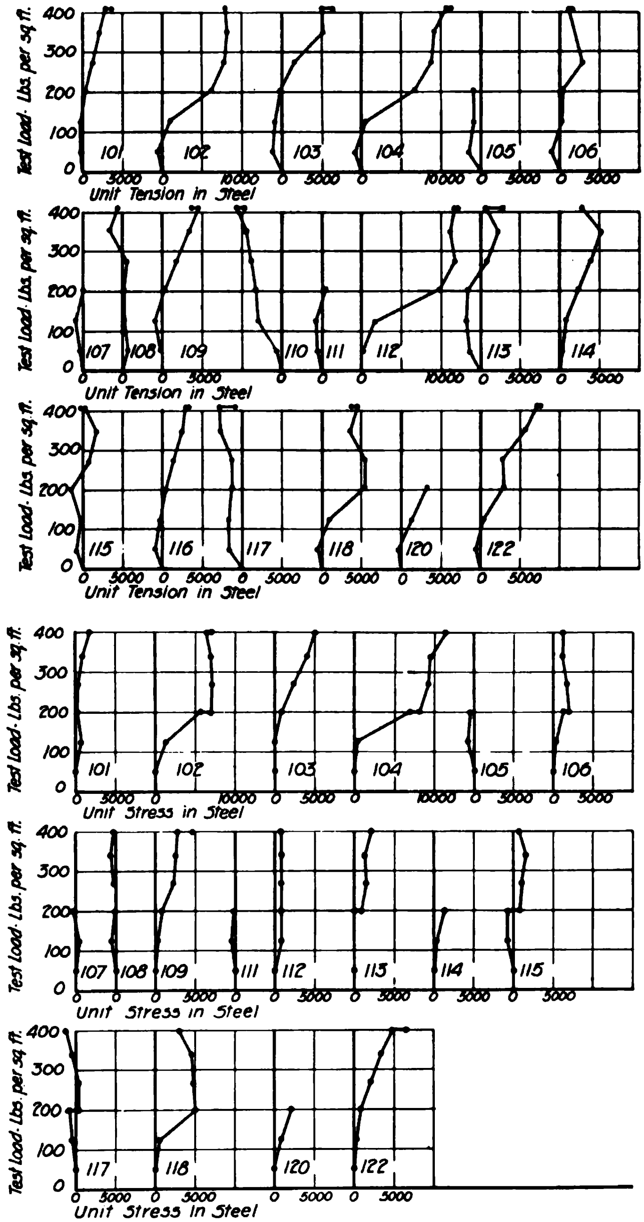


FIG. 51.—LOAD-DEFORMATION DIAGRAMS FOR REINFORCEMENT.

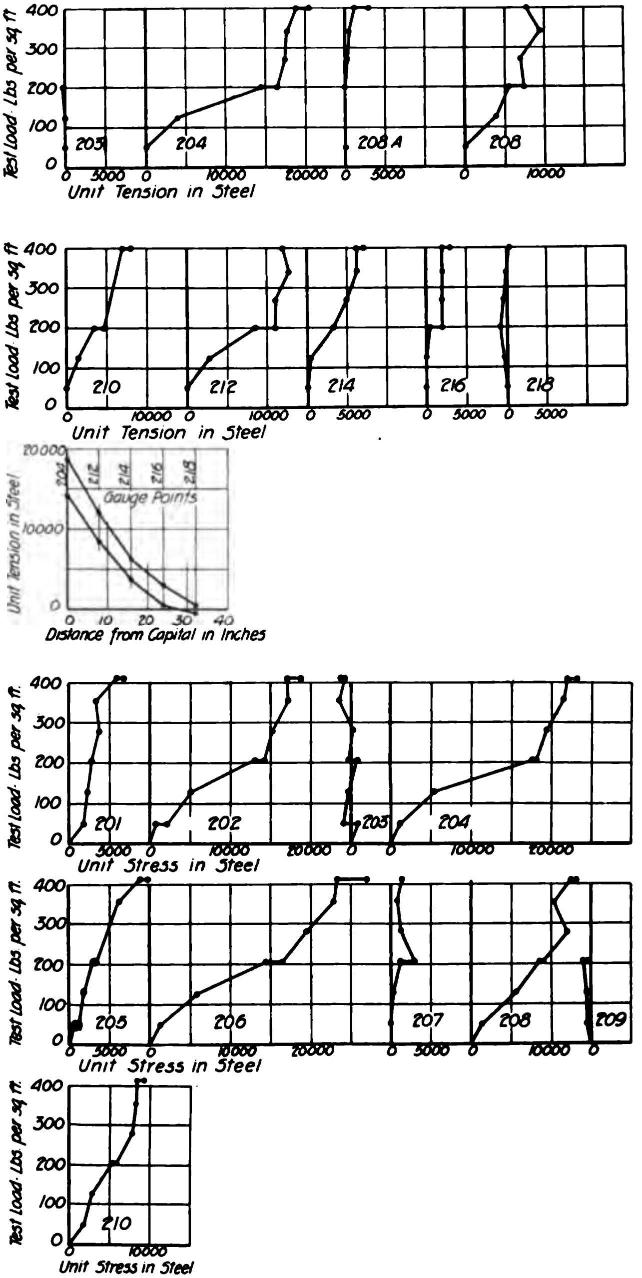


FIG. 52.—LOAD-DEFORMATION DIAGRAMS FOR REINFORCEMENT.
READINGS AND CURVES BY W. A. SLATER.

to trace, and were only of such magnitude as would be expected to accompany the reinforcement stresses observed.

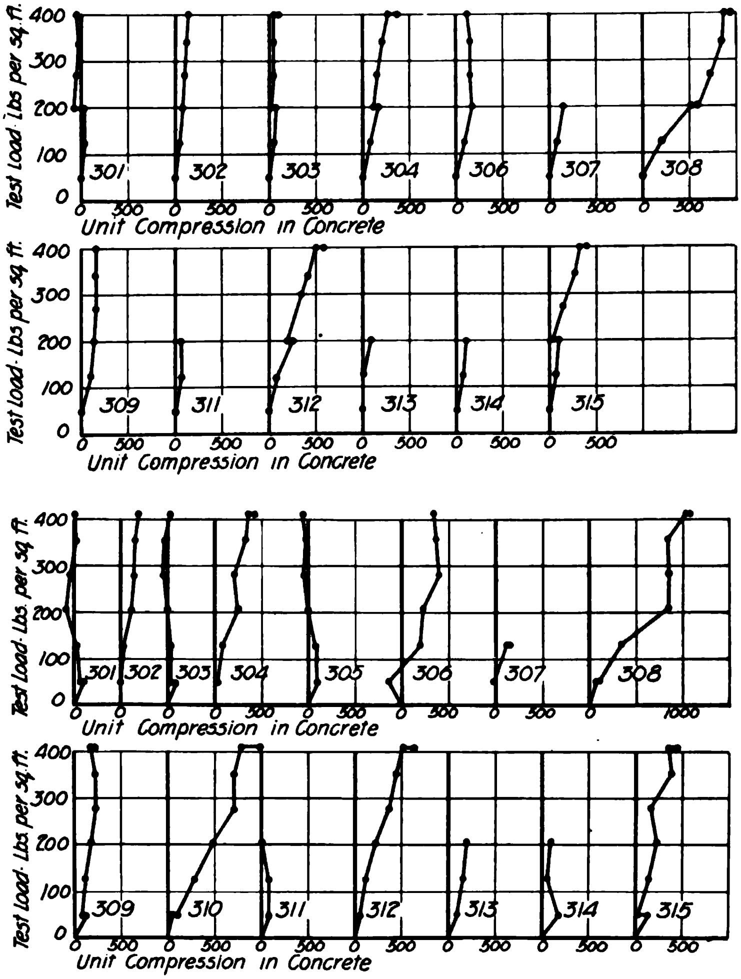


FIG. 53.—LOAD-DEFORMATION DIAGRAMS FOR CONCRETE.

Examination of the plotted deflections and stresses shows a change of inclination of the curves at a load of 200 lb. per sq. ft. Take, for instance, point 202, which is typical of this condition.

At 200 lb. per sq. ft. the stress in this bar was 14,000 lb. per sq. in.; the load producing this stress was that of stage 3.

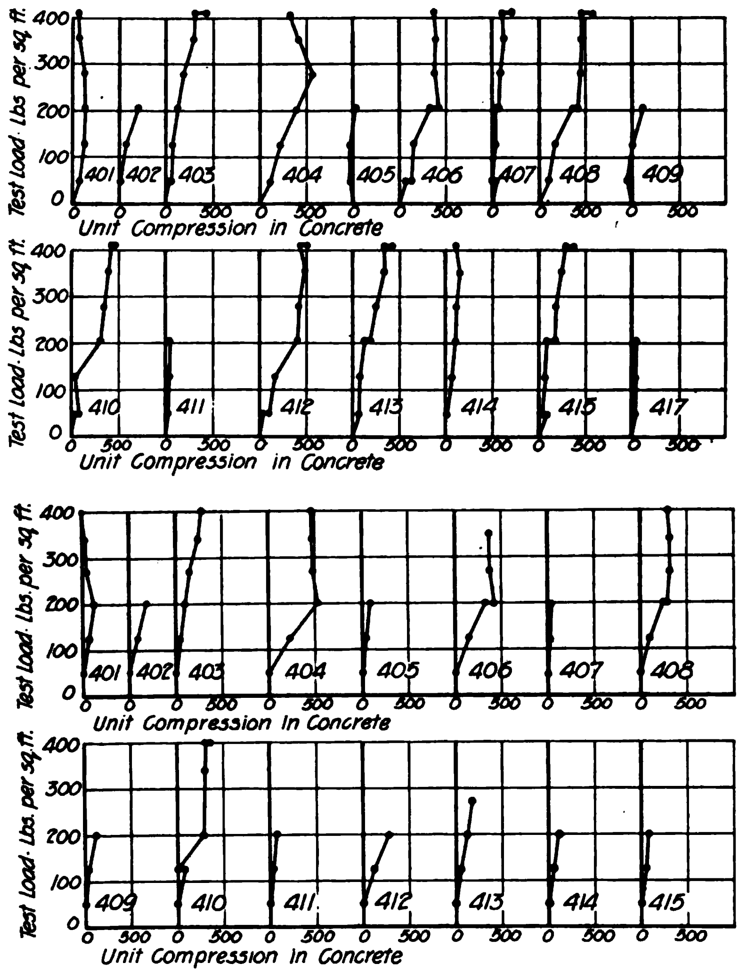


FIG. 54.—LOAD-DEFORMATION DIAGRAMS FOR CONCRETE. READINGS AND CURVES BY W. A. SLATER.

When the load from the adjoining panels was shifted and corresponded to 400 lb. per sq. ft., the stress was only 19,000 lb. per sq. in., instead of some 32,000 obtained by producing the curve to

the 400 lb. line at that slope which obtained below. This indicates that a part of the load is carried by the panels adjoining the loaded one.

The same result is indicated in the case of point 104, which is at the bottom of the slab near the middle of one of the panels. The fact that the adjoining panels assist in carrying a load placed

TABLE XII.—SUMMARY OF STRESSES.

Location.		Gauge Point.	Design Load, 200 lb. per sq. ft.			400 lb. per sq. ft.	
			D. L.*	L. L.	Total.	L. L.	Total.
Reinforcement Stresses.							
Over Column Head	Long Span	202	2,000	14,000	16,000	19,000	21,000
		204	2,500	17,500	20,000	23,000	25,500
		Av.	2,250	15,750	18,000	21,000	23,250
	Short Span	201	1,500	3,000	4,500	7,000	8,500
		205	1,000	3,000	4,000	1,000	11,000
		Av.	1,250	3,000	4,250	8,500	9,750
Bottom of Slab at Side of Panel	Long Span	102	1,000	7,000	8,000	9,000	10,000
		112	1,500	11,500	13,000	13,500	15,000
		Av.	1,250	9,250	10,500	11,250	12,500
	Short Span	101	500	1,000	1,500	4,000	4,500
Bottom of Slab at Middle of Panel	Long Span	104	500	7,500	8,000	12,000	12,500
		122	1,000	4,000	5,000	9,500	10,500
		Av.	750	5,750	6,500	10,750	11,500
	Short Span	103	500	1,500	2,000	8,000	8,500
		109	1,000	2,500	3,500	6,500	7,500
		Av.	750	2,000	2,750	7,250	8,000
Concrete Stresses.							
Bottom of Slab at Column	Long Span	308	100	750	850	1,050	1,150
	Short Span	315	100	250	350	450	550
Top of Slab at Edge of Panel	Long Span	406	100	300	400	350	450
		412	100	400	500	500	600
		Av.	100	350	450	425	525
Top at Middle	Long Span	408	150	400	550	600	750
Bottom of Slab at Wall	Long Span	302	50	150	200	250	300
		304	100	250	350	400	500
		Av.	75	200	275	325	400

* Dead-load stresses obtained from stress-deformation curves by projecting curve down to no-load line.

on one panel is also shown by the fact that deflections were observed in panels adjacent to those loaded, as shown in Fig. 48 and by the fact that cracks on the under side of the slab were traceable well past the middle of panels adjoining these loads.

The point of contraflection was found to be at 0.21 of the clear span from the edge of the column head.

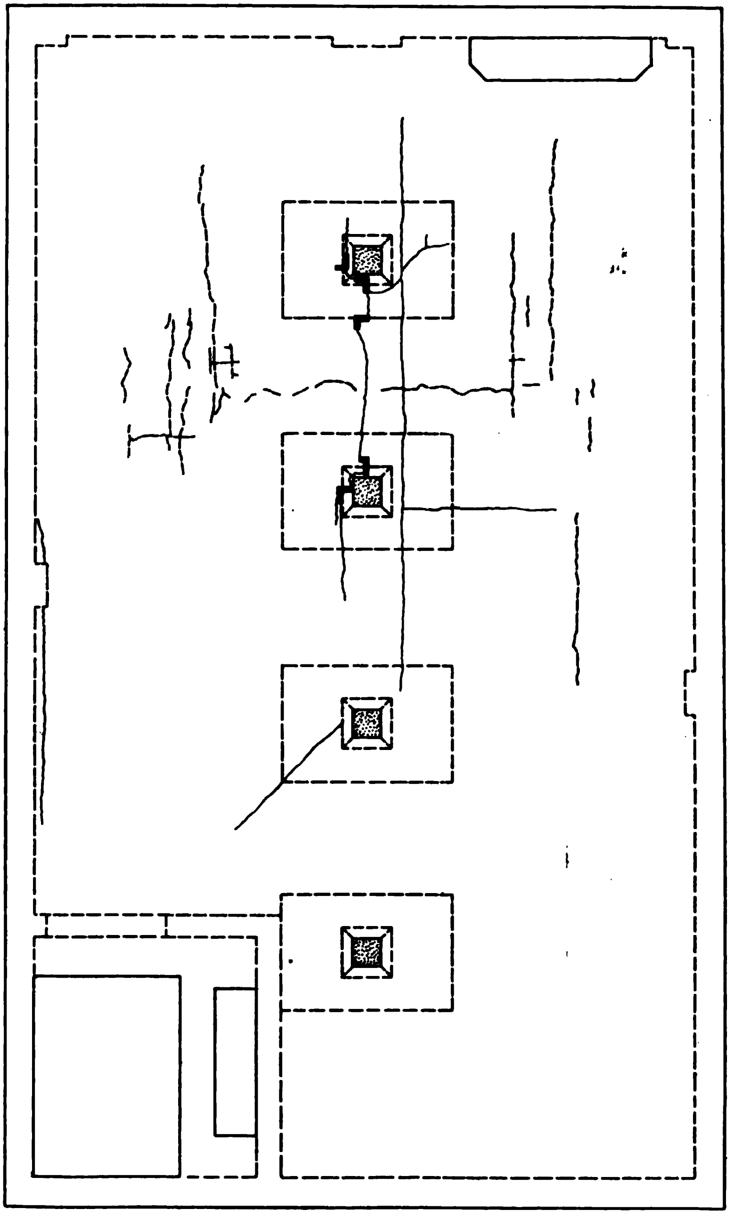


FIG. 55.—PLAN SHOWING LOCATION OF CRACKS. FULL LINES INDICATE CRACKS ON TOP AND DOTTED LINES ON BOTTOM OF SLAB.

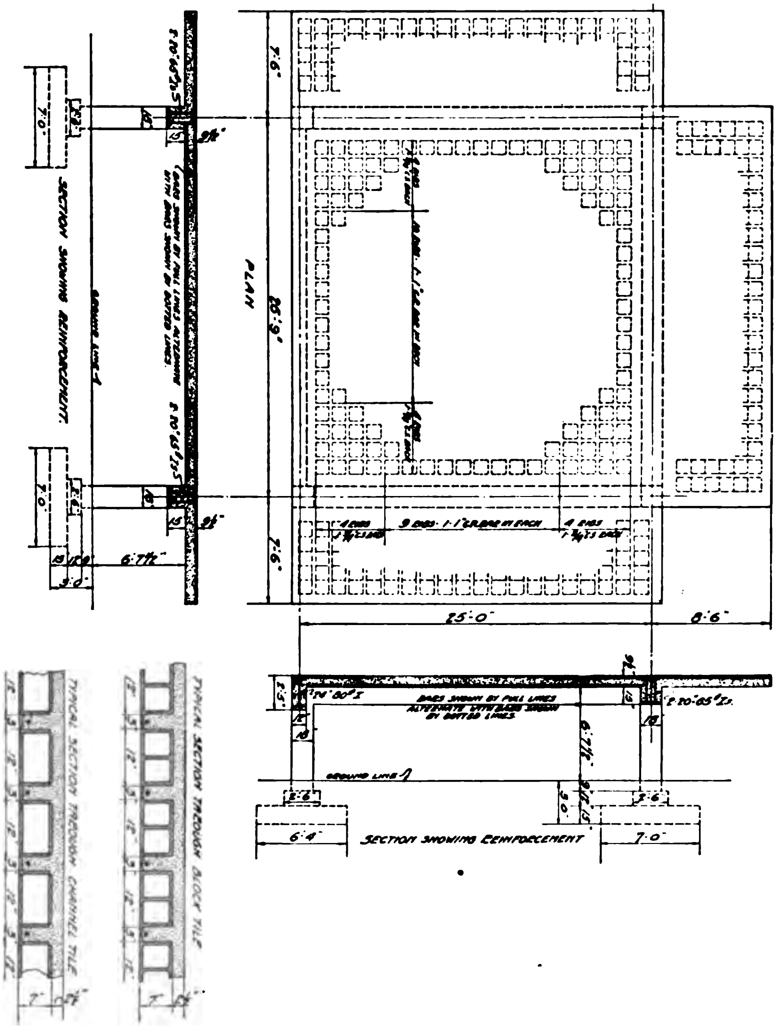


FIG. 56.—BARR BUILDING TEST PANEL, ST. LOUIS, MO.

The high values of bond stress observed indicate that consideration be given to this feature in the design of flat slabs, especially when it is realized that no matter what scheme of reinforcement be adopted there will exist stresses at right angles to any given bar over the column which tend to destroy the adhesion of the concrete to the reinforcement.

PART III. TEST OF A CONCRETE FLOOR REINFORCED IN TWO DIRECTIONS.

Test Panel.—The test was made on a panel representing a wall panel of a continuous floor and to approximate existing conditions, cantilevers were built on 3 sides. These cantilevers were loaded

FIG. 57.—BARR BUILDING TEST PANEL WITH LOAD OF 380 LB. PER SQ. FT.

during the test until the tangents to the slab over the supporting beams became approximately horizontal, as would be the case in a continuous floor under multiple panel loading.

The panel proper is 25 ft. long by 26 ft. 9 in. wide, Fig. 56; and is carried by steel I beams, fireproofed with concrete. These beams rest on steel bearing plates on concrete posts at each corner of the panel. Seven-inch tile was used with a cover of 2½ in. of 1:1½:3 concrete, making the total thickness of the slab 9½ in.

The arrangement of tile is such that when alternate rows of block-tile and channel-tile are laid, the concrete is formed into two series of intersecting T beams, spaced 15 in. centers each way and having 3-in. stems. Small furring-tile, laid at the intersection

of the ribs thus formed, complete the tile ceiling surface. The panel was reinforced as shown in Fig. 56, designed for a live load of 150 lb. per sq. ft.

The panel was poured on September 1, 1911, under unfavorable conditions, it being necessary to carry on a considerable portion

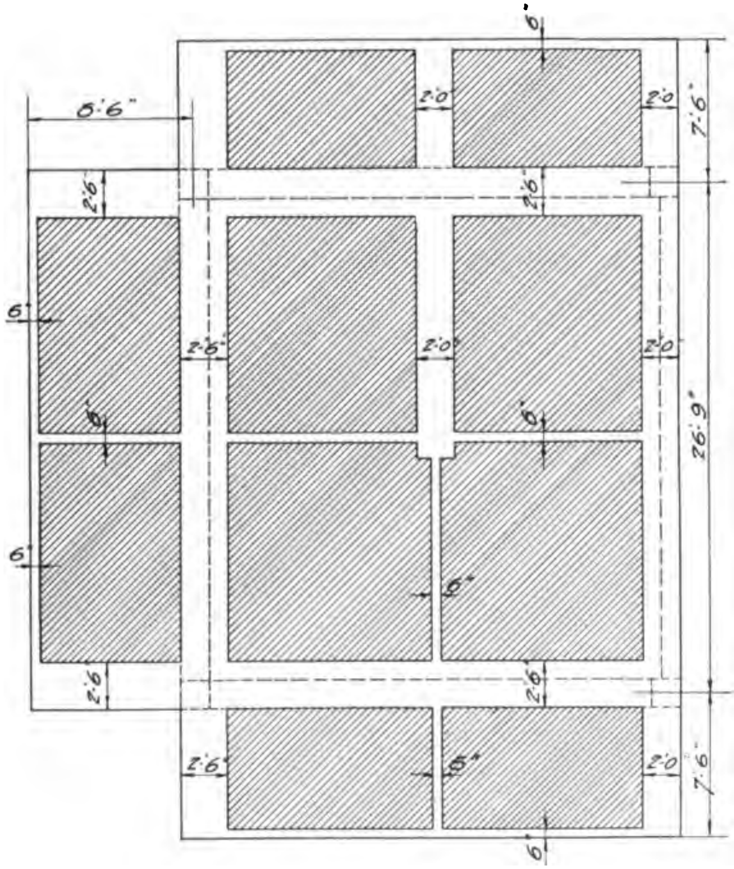


FIG. 58.—PLAN SHOWING METHOD OF LOADING BARR BUILDING TEST PANEL.

of the work after dark. The weather was very hot during the next few days and several of them were holidays, so that the slab was not even kept wet. On the whole, conditions under which this slab was erected were probably not more favorable than would exist in an actual building.

Method of Testing.—Sand in sacks was used for the load, Fig. 57, each sack being filled on the scales to weigh 100 lb. The loading was started on November 23, 1911, and progressed by stages, alternating with readings, until 650 lb. per sq. ft. was reached on December 6, 1911. The test was stopped at this point because of the expense involved in placing the load upon the high piles of sacks. The total load remained in place on the panel for 48 hours, after which unloading was started and completed 5 days later.

TABLE XIII.—STAGES OF LOADING.

Stage.	Time of Loading.		Load in lb. per sq. ft.			
	Start.	Finish.	Panel.	Cantilever.		
				Left.	Rear.	Right.
	1911	1911				
1	Nov. 23, 2.30 P. M.	Nov. 24, 10.00 A. M.	75.5	38.3	38.6	39.4
2	Nov. 25, 11.00 A. M.	Nov. 25, 3.30 P. M.	75.5	74.3	70.3	74.3
3	Nov. 27, 2.00 P. M.	Nov. 27, 4.00 P. M.	75.5	93	93.2	93
4	Nov. 28, 12.30 P. M.	Nov. 28, 6.30 P. M.	150	183.5	194.2	191.5
5	Nov. 29, 12.30 P. M.	Nov. 29, 5.20 P. M.	247	183.5	194.2	191.5
6	Nov. 30, 1.00 P. M.	Nov. 30, 4.50 P. M.	300	183.5	247	191.5
7	Dec. 1, 10.00 A. M.	Dec. 1, 2.10 P. M.	300	300	298	300
8	Dec. 1, 4.30 P. M.	Dec. 1, 5.15 P. M.	300	346	298	346
9	Dec. 2, 9.30 A. M.	Dec. 2, 9.50 A. M.	300	372	298	372
10	Dec. 2, 10.00 A. M.	Dec. 2, 10.15 A. M.	300	385	298	385
11	Dec. 2,	Dec. 2, 12.05 P. M.	300	385	298	385
12	Dec. 3, 9.00 A. M.	Dec. 3, 9.45 A. M.	360	385	298	385
13	Dec. 3, 9.00 A. M.	Dec. 3, 2.00 P. M.	380	465	376	465
14	Dec. 5, 10.30 A. M.	Dec. 5, 4.00 P. M.	500	465	376	465
15	Dec. 6, 9.40 A. M.	Dec. 6, 10.40 A. M.	527	465	376	465
16	Dec. 6, 11.20 A. M.	Dec. 6, 12.15 P. M.	550	465	376	465
17	Dec. 6, 12.50 P. M.	Dec. 6, 1.35 P. M.	567	465	376	465
18	Dec. 6, 2.10 P. M.	Dec. 6, 3.00 P. M.	591	465	376	465
19	Dec. 7, 8.00 A. M.	Dec. 7, 9.20 A. M.	615	465	376	465
20	Dec. 7, 10.50 A. M.	Dec. 7, 1.10 P. M.	650	465	376	465

Both reinforcement and concrete stresses, as well as deflections, were measured throughout the test, exceptionally complete sets of readings being taken under the design load of 150 lb. per sq. ft. and under the test load required by the City of St. Louis, *i. e.*, a superimposed load equal to once the dead load plus twice the live load, or 380 lb. per sq. ft.

Deflections were read by measuring the distance between a steel plate fastened to the ceiling and a steel rod held in a scaffold below. An inside micrometer reading to 0.001 in. was used, both plate and rod having countersunk holes to locate exactly the position of the instrument.

The stresses in the reinforcement were read by an extensometer which is a modification of the Berry Strain Gauge used to read stresses on the concrete, while W. A. Slater read reinforcement stresses with a modification of the Berry Strain-Gauge, made at

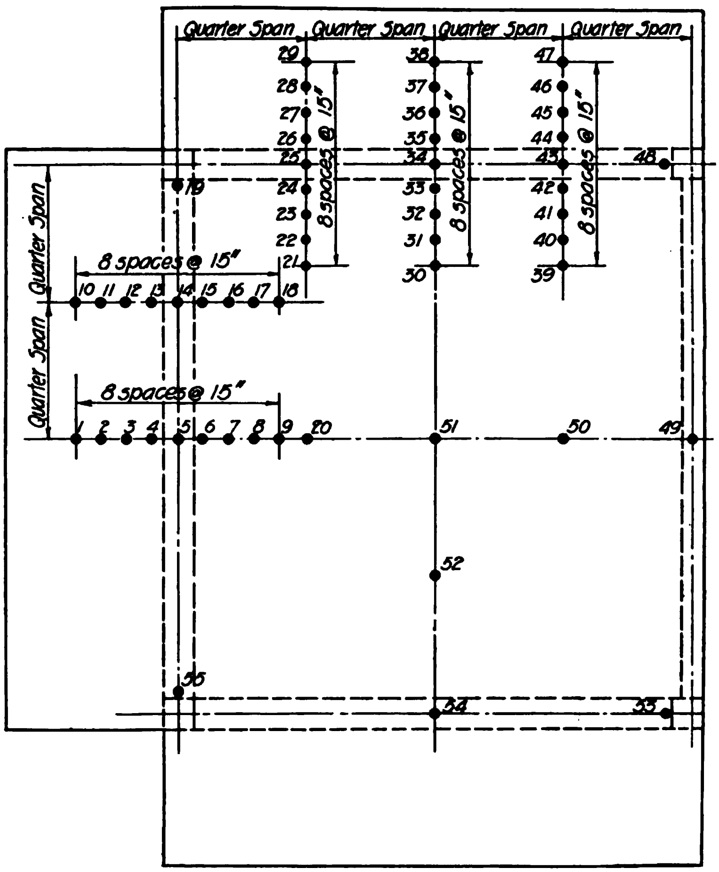


FIG. 59.—PLAN SHOWING LOCATION OF DEFLECTION POINTS.

the University of Illinois, Figs. 1 and 2. Both of these instruments have points which are inserted into small holes drilled in the steel or in metal plugs set in the concrete and read directly to 1/5000 of an inch, or by estimation to one quarter of this amount. An

TABLE XIV.—DEFLECTIONS, BARR BUILDING TEST PANEL.

Deflection Point.	Unit Load.	75.5	150	300	300	380	380	500	500	527	550	567	591	615	650
	Stage.	3	4	6	11	13	13	14	14	15	16	17	18	19	20
	Date and Hour.	Nov. 27 4 P. M.	Nov. 29 11 A. M.	Nov. 30 5 P. M.	Dec. 2 12.30 P. M.	Dec. 3 2.30 P. M.	Dec. 5 9 A. M.	Dec. 5 4 P. M.	Dec. 6 9 A. M.	Dec. 6 11 A. M.	Dec. 6 12.30 P. M.	Dec. 6 1.45 P. M.	Dec. 6 3.10 P. M.	Dec. 7 9.30 A. M.	Dec. 7. 2.30 P. M.
1	....	.....	.354	.....	.903	1.340	1.417	1.456	.....	.....	.....	.....	.....	1.456	.....
2	....	.....	.266	.....	.724	1.043	1.099	1.216	.....	.....	.....	.....	.....	1.216	.....
3	....	.....	.198	.....	.559	.794	.831	.....	.....	.....	.....	.....	.....	1.017	.....
4	....	.....	.153	.....	.428	.591	.611	.....	.....	.....	.....	.....	.....	.870	.....
5	....	.....	.130	.....	.420	.567	.581	.....	.....	.....	.....	.....	.....	.952	.....
6	....	.....	.117	.....	.421	.558	.568	.....	.....	.....	.....	.....	.....	1.167	.....
7	....	.....	.110	.....	.480	.645	.658	.....	.....	.....	.....	.....	.....	1.332	.....
8	....	.....	.127	.....	.568	.764	.778	.....	.....	.....	.....	.....	.....	1.631	.....
9	....	.....	.142	.....	.645	.878	.897	.....	.....	.....	.....	.....	.....	1.921	.....
20	....	.....	.134	.....	.693	.960	.982	.....	.....	.....	.....	.....	.....	2.155	.....
10	....	.....	.296	.....	.860	1 153	1.222	.....	.....	.....	.....	.....	.....	.....	.....
11	....	.....	.213	.....	.598	.890	.940	.....	.....	.....	.....	.....	.....	.....	.....
12	....	.....	.160	.....	.438	.636	.668	.....	.....	.....	.....	.....	.....	.....	.....
13	....	.....	.104	.....	.302	.419	.438	.....	.....	.....	.....	.....	.....	.....	.....
14	....	.....	.094	.....	.317	.424	.434	.....	.....	.....	.....	.....	.....	.....	.....
15	....	.....	.089	.....	.310	.410	.417	.....	.....	.....	.....	.....	.....	.....	.....
16	....	.....	.078	.....	.352	.468	.478	.....	.....	.....	.....	.....	.....	.....	.....
17	....	.....	.086	.....	.408	.546	.558	.....	.....	.....	.....	.....	.....	.....	.....
18	....	.....	.090	.....	.460	.622	.633	.....	.....	.....	.....	.....	.....	.....	.....
19	Col.	.02	.091	.191	.067	.073	.078	.094	.100	.103	.107	.109	.114	.126	.137
21	....	.....	.071	.....	.402	.544	.553	.....	.....	.....	.....	.....	.....	.....	.....
22	....	.....	.054	.....	.331	.444	.453	.....	.....	.....	.....	.....	.....	.....	.....
23	....	.....	.043	.....	.265	.354	.358	.....	.....	.....	.....	.....	.....	.....	.....
24	....	.....	.034	.....	.205	.268	.274	.....	.....	.....	.....	.....	.....	.....	.....
25	....	.....	.024	.....	.172	.234	.247	.....	.....	.....	.....	.....	.....	.....	.....
26	....	.....	.019	.....	.148	.228	.239	.....	.....	.....	.....	.....	.....	.....	.....
27	....	.....	.022	.....	.154	.271	.290	.....	.....	.....	.....	.....	.....	.....	.....
28	....	.....	.025	.....	.165	.330	.355	.....	.....	.....	.....	.....	.....	.....	.....
29	....	.....	.030	.....	.179	.394	.431	.....	.....	.....	.....	.....	.....	.....	.....
30	....	.....	.116	.....	.530	.725	.742	.....	.....	.....	.....	.....	.....	.....	.....
31	....	.....	.098	.....	.448	.603	.616	.....	.....	.....	.....	.....	.....	.....	.....
32	....	.....	.082	.....	.362	.477	.488	.....	.....	.....	.....	.....	.....	.....	.....
33	....	.....	.075	.....	.293	.376	.384	.....	.....	.....	.....	.....	.....	.....	.....
34	....	.....	.068	.....	.262	.351	.361	.....	.....	.....	.....	.....	.....	.....	.....
35	....	.....	.073	.....	.244	.351	.366	.....	.....	.....	.....	.....	.....	.....	.....
36	....	.....	.068	.....	.238	.373	.392	.....	.....	.....	.....	.....	.....	.....	.....
37	....	.....	.072	.....	.204	.415	.442	.....	.....	.....	.....	.....	.....	.....	.....
38	....	.....	.082	.....	.248	.467	.502	.....	.....	.....	.....	.....	.....	.....	.....
39	....	.....	.071	.....	.386	.527	.545	.....	.....	.....	.....	.....	.....	.....	.....
40	....	.....	.063	.....	.324	.432	.446	.....	.....	.....	.....	.....	.....	.....	.....
41	....	.....	.054	.....	.260	.338	.347	.....	.....	.....	.....	.....	.....	.....	.....
42	....	.....	.049	.....	.202	.251	.258	.....	.....	.....	.....	.....	.....	.....	.....
43	....	.....	.046	.....	.177	.228	.238	.....	.....	.....	.....	.....	.....	.....	.....
44	....	.....	.053	.....	.159	.232	.245	.....	.....	.....	.....	.....	.....	.....	.....
45	....	.....	.064	.....	.167	.263	.285	.....	.....	.....	.....	.....	.....	.....	.....
46	....	.....	.079	.....	.186	.324	.350	.....	.....	.....	.....	.....	.....	.....	.....
47	....	.....	.095	.....	.202	.380	.414	.....	.....	.....	.....	.....	.....	.....	.....
48	Col.	.003	.016	.033	.047	.066	.068	.085	.087	.094	.095	.099	.103	.107	.115
49	....	.027	.042	.118	.119	.155	.158	.264	.274	.294	.324	.348	.379	.414	.469
50	....	.....	.104	.....	.643	.900	.926	.....	.....	.....	.....	.....	.....	.....	.....
51	Center	.129	.274	.774	.925	1.299	1.331	1.947	2.019	2.153	2.327	2.444	2.658	2.897	3.313
52	....	.....	.124	.....	.725	.993	1.017	.....	.....	.....	.....	.....	.....	.....	.....
53	Col.	.009	.229	.152	.127	.148	.154	.162	.175	.175	.171	.170	.175	.190	.196
54	....	.040	.013	.211	.335	.434	.449	.535	.545	.561	.586	.601	.627	.659	.702
55	Col.	.039	.071	.114	.078	.105	.113	.130	.136	.139	.143	.147	.150	.158	.165
....	....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
....	....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
51	Corr	0.080	.167	.594	.641	.920	.944	1.459	1.516	1.632	1.777	1.872	2.052	2.252	2.608

Last line of readings are deflections at center of panel assuming supporting beams to be rigid. All other readings corrected for pier settlement only.

8 in. gauge length was used throughout the test. The length of such an instrument varies with changes of the temperature of the observer's hands, to correct for which readings were taken at intervals on standard bars and all temperature corrections necessary have been applied to the stresses given in the tables and plotted in the curves.

Method of Loading.—To prevent the possibility of the load arching, the sacks of sand were arranged in separate piles, as

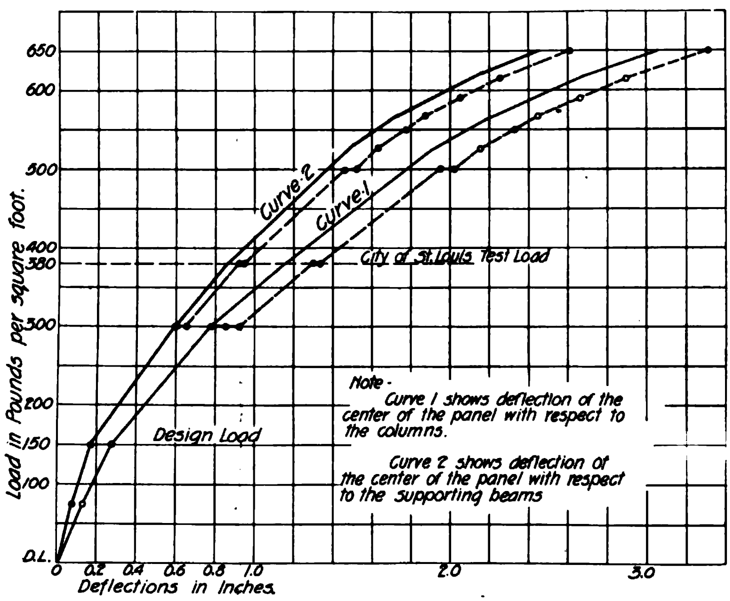


FIG. 60.—DEFLECTIONS AT CENTER OF PANEL.

shown in Fig. 57. The wide aisles, Fig. 58, were necessary for the accommodation of the instruments and observers. Table XIII gives the loading on the various portions of the panel at different times, as well as the loads per square foot of slab and cantilevers and the date and hour at which each stage was started and completed. The loads per square foot are in each case obtained by dividing the total load on the panel or cantilever by the area, the intensity of load under the piers being much greater.

Up to the test load required by the City of St. Louis the

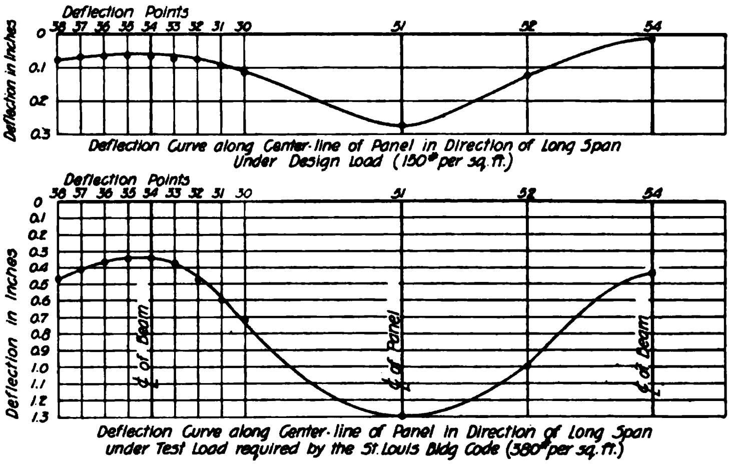


FIG. 61.—DEFLECTION CURVE IN DIRECTION OF LONG SPAN.

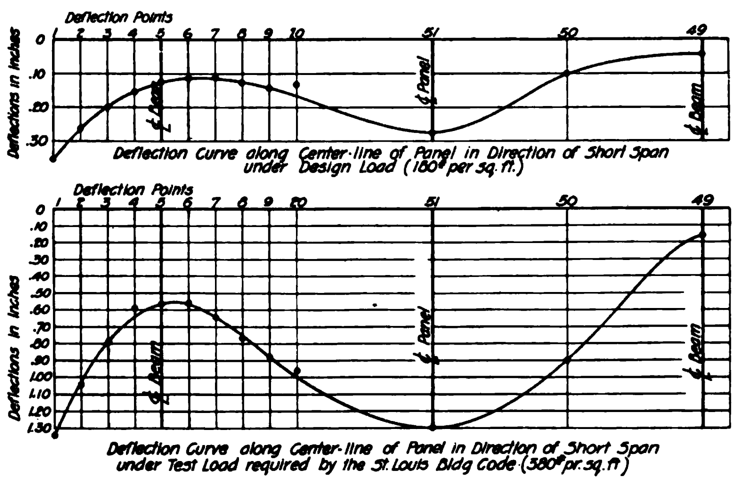


FIG. 62.—DEFLECTION CURVE IN DIRECTION OF SHORT SPAN.

cantilevers were carefully loaded to horizontality, but from this stage on no more load was applied to the cantilevers and they rose slightly, thus increasing the deflection at the center of the panel.

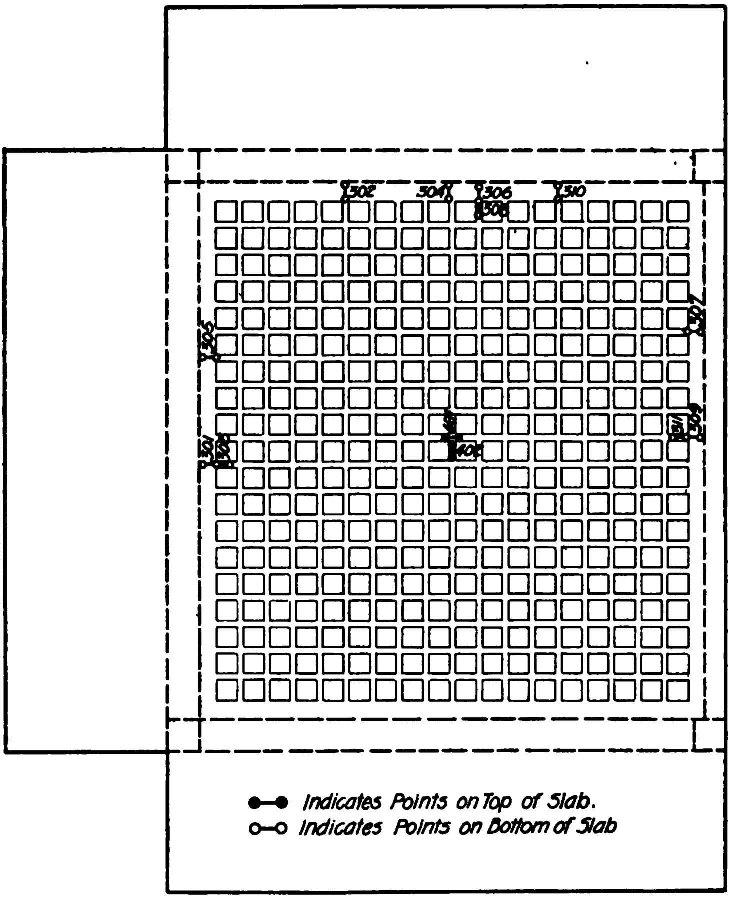


FIG. 63.—PLAN SHOWING LOCATION OF GAUGE POINTS ON CONCRETE.

The horizontality of the tangents over the beams was determined by measuring deflections of both the slab and cantilevers at a series of points along a line perpendicular to the beams. The deflections were then plotted and a smooth curve drawn

which showed at a glance whether or not horizontality existed. It was often found necessary to shift the load on the cantilevers several times before the correct amount and position were reached.

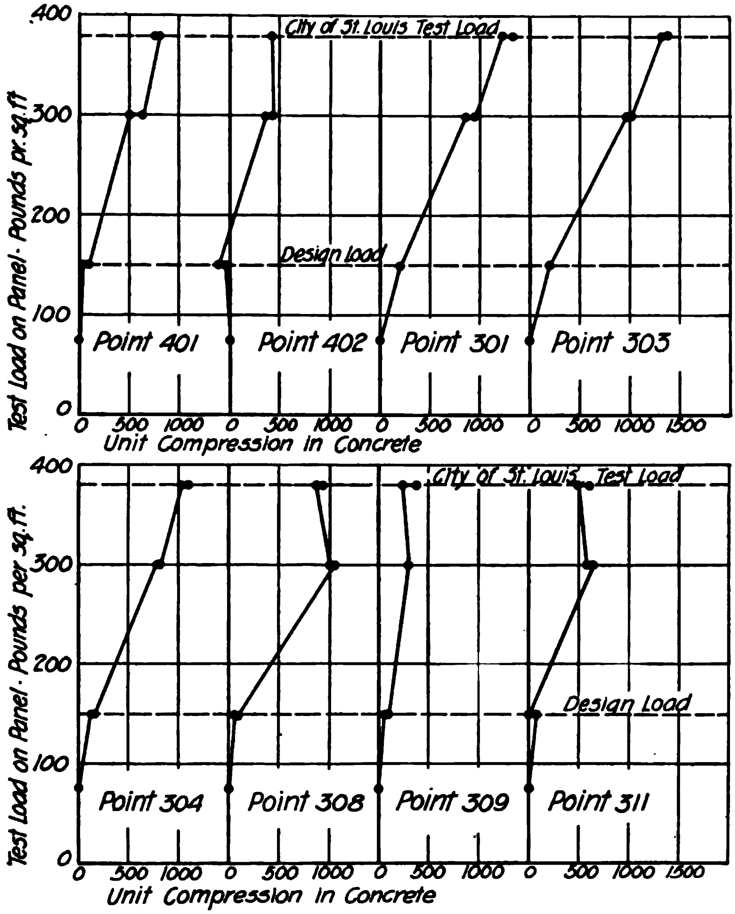


FIG. 64.—DIAGRAM OF STRESSES IN CONCRETE.

Deflections.—Deflections were read at the points shown in Fig. 59. A summary of the observations is given in Table XIV. There was a settlement of the footings which has been corrected for in the summary and in the curves.

The center point deflections are plotted in Fig. 60. The deflections shown by the curves represent the deflections of the panel with respect to the supporting beams. In order to arrive at the deflection of the panel proper, the gross deflections were decreased by an amount equal to the average deflection of

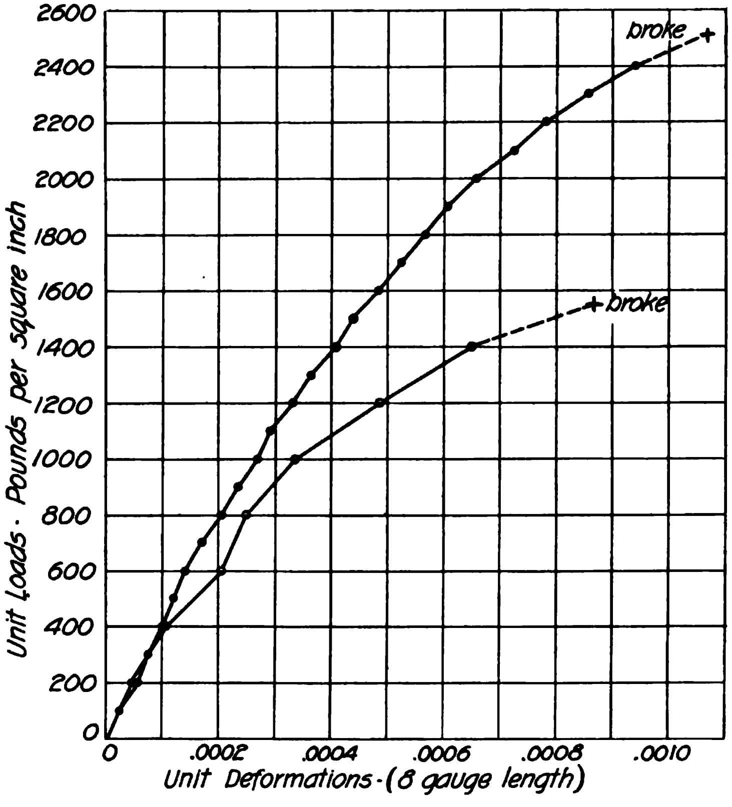


FIG. 65.—STRESS-STRAIN DIAGRAM OF TWO CONCRETE TEST CYLINDERS.

the supporting beams. Curve 1 shows the gross deflections, the dotted curve including settlements occurring during the intervals between loads and the solid line giving the true elastic deflections (including beam deflections) caused by a uniform rate of loading. The dotted portion of Curve 2 shows the same thing

after correcting to a uniform rate of loading. The solid line, Curve 2, should be used in comparing this test with others or as a basis for deflection coefficients.

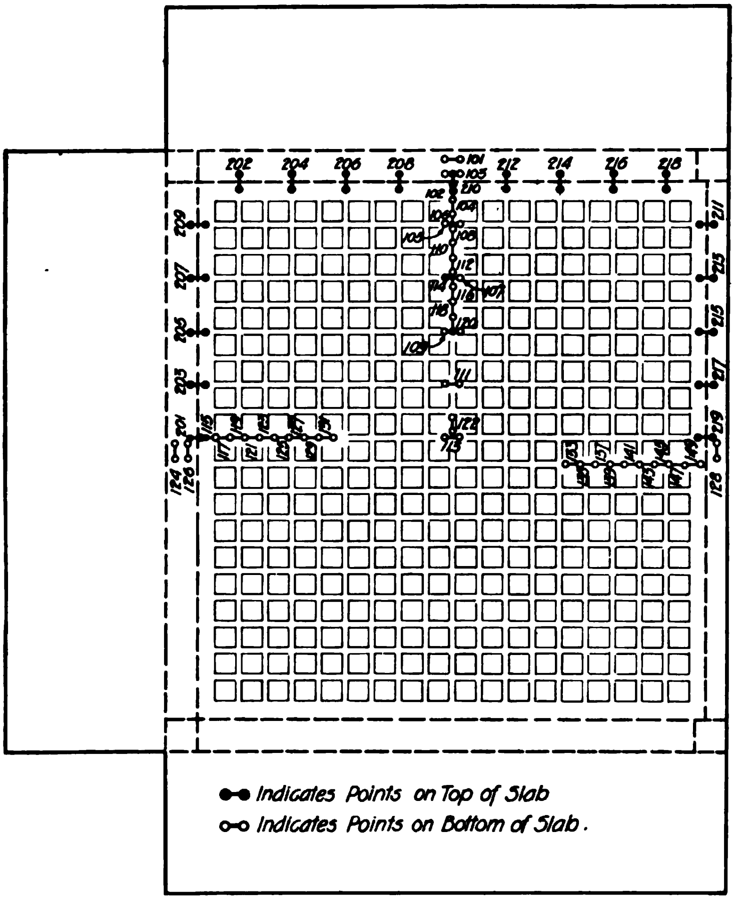


FIG. 66.—PLAN SHOWING LOCATION OF GAUGE POINTS ON REINFORCEMENT.

Fig. 61 shows exaggerated scale sections of the panel along the long axis for the design load and for the load specified by the St. Louis Building Laws. The deflections, from which these are plotted, are corrected for settlement of the footings only. Fig. 62

shows similar sections along the short axis of the panel. Inspection of these figures will show that the cantilevers were properly loaded to obtain horizontal tangents over the supporting beams.

Deformations and Stresses.—Concrete stresses were read at the points shown in Fig. 63. A summary of the observed stresses is given in Table XV; the load-stress curves at a few of these points are plotted in Fig. 64. In these curves and in the summary, the stresses are those which were induced in the concrete by the loads greater than 75 lb. per sq. ft. At this stage the points, at which these stresses were read, worked loose and had to be reset.

TABLE XV.—CONCRETE STRESSES, BARR BUILDING TEST PANEL.

Gauge Point.	Unit Load.	150	150	150	300	380	380
	Stage.	4	4	4	11	13	13
	Date and Hour.	Nov. 28 7 P. M.	Nov. 29 10 A. M.	Nov. 29 12 M.	Dec. 2 12 M.	Dec. 3 3.30 P. M.	Dec. 4 9.30 P. M.
401	R. O.	−40	−285	−81	−625	−796	−755
402	R. O.	+40	−162	+122	−430	−422	−422
301	R. O.	−195	−495	−203	−943	−1225	−1322
303	R. O.	−195	−504	−211	−1005	−1322	−1370
305	R. O.	−105	−350	−57	−740	−935	−1022
302	R. O.	−25	−358	−57	−723	−870	−967
304	R. O.	−122	−382	−162	−796	−1022	−1095
308	R. O.	−58	−252	−73	−1005	−862	−926
310	R. O.	+65	−219	+130	−740	−796	−943
307	R. O.	+146	−471	+90	+130	+365	+211
309	R. O.	−49	−398	−81	−292	−220	−374
311	R. O.	−80	−325	+0	−560	−480	−600

+ = Tension.
− = Compression.

The reduction in stress with increasing load appearing in some of the curves may be the result of the gradual giving away of the concrete in tension and resulting shifting of the point of inflection.

Two test cylinders 8 x 15 in. were taken from the mixer at the time the slab was poured and after being cured under the same conditions as the panel itself, they were tested just after completion of the readings on the panel. The ultimate strength and the modulus of elasticity of these were widely different as can be seen from the stress-strain diagrams in Fig. 65. In deriving the concrete stresses the average modulus was used, being taken at 3,250,000.

TABLE XVI.—REINFORCEMENT STRESSES, BARR BUILDING TEST PANEL.

Gauge Point.	Unit Load.	75.5	150	150	300	380	380	500	527	550	567	591	615	650	650
	Stage.	2	4	4	11	13	13	14	15	16	17	18	19	20	20
	Date and Hour.	Nov. 25 4 A. M.	Nov. 28 7 P. M.	Nov. 29 10 A. M.	Dec. 2 12 M.	Dec. 3 3 P. M.	Dec. 5 10 A. M.	Dec. 5 4.30 P. M.	Dec. 6 11 A. M.	Dec. 6 12.30 P. M.	Dec. 6 2 P. M.	Dec. 6 3 P. M.	Dec. 7 10 A. M.	Dec. 7 3.30 P. M.	Dec. 8 9.30 A. M.
201	......	+300	+6,050	+6,750	+16,600 +16,800	+28,500 +27,600	+28,500	+31,900	+32,900	+34,400	......	+34,500 +34,050	+37,400 +35,500	+37,700	......
203	......	−75	+5,000	+4,875	+16,100 +15,400	+27,600 +29,000	......	......	......	......	......	......	......	......	......
205	......	−1,275	+2,325	+2,175	+12,200 +12,100	+25,100 +25,000	......	......	......	......	......	......	......	......	......
207	......	−675	+5,000	+5,000	+18,600 +18,300	+31,600 +32,100	......	......	......	......	......	......	......	......	......
209	......	−600	+4,125	+5,850	+18,900 +18,900	+38,600 +36,600	+31,700	+39,200	......	......	......	......	......	......	......
202	......	−225	+1,950	+525	+6,400 +5,300	+18,200 +17,300	+18,000	+15,600	......	......	......	......	+29,600	+31,800	......
204	......	−1,050	......	+3,000	+15,200 +14,200	+25,100 +26,500	......	......	......	......	......	......	......	......	......
206	......	−1,050	+1,800	+1,875	+16,900 +16,300	+32,500 +31,000	......	......	......	......	......	......	......	+78,800	......
208	......	−750	+2,400	+3,300	+23,900 +22,100	+39,700 +38,500	......	+57,100	+56,900	+62,400	......	+67,500 +66,200	+71,800 +69,400	......	......
210	......	−1,125	+2,700	+3,775	+20,500 +19,100	+35,300 +35,000	+37,400	+47,250	+51,400	+61,500	......	+61,800 +59,200	+68,600 +67,000	+68,600 +38,100	......
212	......	−1,500	......	+2,250	+23,700 +15,100	+37,800 +33,800	......	......	......	......	......	......	......	......	......

+ = Tension.
− = Compression.

TABLE XVI—REINFORCEMENT STRESSES, BARR BUILDING TEST PANEL.—*Continued.*

Gauge Point.	Unit Load.	75.5	150	150	300	380	380	500	527	550	567	591	615	650	650
	Stage.	2	4	4	11	13	13	14	15	16	17	18	19	20	20
	Date and Hour.	Nov. 25 4 A. M.	Nov. 28 7 P. M.	Nov. 29 10 A. M.	Dec. 2 12 M.	Dec. 3 3 P. M.	Dec. 5 10 A. M.	Dec. 5 4.30 P. M.	Dec. 6 11 A. M.	Dec. 6 12.37 P. M.	Dec. 6 2 P. M.	Dec. 6 3 P. M.	Dec. 7 10 A. M.	Dec. 7 3.30 P. M.	Dec. 8 9.30 A. M.
214		−750	+1,425	+1,725	+22,000 +18,400	+38,000 +36,700									
216		−750	+1,275	+1,125	+14,600	+24,200									
218		−2,250		+300	+7,400 +4,700	+16,100 +14,000	+16,400	+20,600					+25,400	+27,500	
211		..0		−675	+4,300 +750	+9,400 +10,500	+7,900	+18,350							
213		−375		+675	+1,800 +300	+1,400 +1,500									
215		−375		+900	+1,650 +375	+1,950 +1,200									
217		−1,125		+675	+1,900 +750	+2,475 +2,300									
219		+6,000	+8,100	+7,800	+8,100 +1,700	+3,400	+8,600	+8,000	+8,850			+8,650 +3,700	+9,900 +1,900	+2,100	
101		+1,350	+6,900	+6,500	+11,700 +10,900	+16,600 +15,100	+15,350	+19,050	+20,100			+20,800	+24,800 +23,600	+26,700	+28,600 +35,800
103		+1,275	+7,800	+6,600	+13,700 +11,300	+19,650 +16,900	+18,750	+23,850	+25,150			+33,300	+30,500 +29,200	+34,100	+35,800
105		−825	+3,750	+2,175	+3,300 +2,000	+8,000 +8,000	+7,300	+9,850				+10,000	+13,000		

+ = Tension.
− = Compression.

TABLE XVI—REINFORCEMENT STRESSES, BARR BUILDING TEST PANEL.—*Continued.*

Gauge Point.	Unit Load.	75.5	150	150	300	380	380	500	527	550	567	591	615	650	650
	Stage.	2	4	4	11	13	13	14	15	16	17	18	19	20	20
	Date and Hour.	Nov. 25 4 A. M.	Nov. 28 7 P. M.	Nov. 29 10 A. M.	Dec. 2 12 M.	Dec. 3 3 P. M.	Dec. 5 10 A. M.	Dec. 5 4.30 P. M.	Dec. 6 11 A. M.	Dec. 6 12.30 P. M.	Dec. 6 2 P. M.	Dec. 6 3 P. M.	Dec. 7 10 A. M.	Dec. 7 3.30 P. M.	Dec 8. 9.30 A. M.
107		+225		+4,200	+7,600 +5,700	+13,700 +4,300						+20,400	+24,200		
109		+1,575	+7,300	+6,150	+11,800 +9,500	+19,300 +18,200						+33,100	+37,700		
111		+1,050	+6,700	+5,550	+15,300 +12,400	+24,300 +22,500						+39,100	+40,800		
113		+1,275	+8,200	+7,050	+17,600 +14,500	+26,100 +24,500	+26,150	+36,750	+40,100	+42,300	+47,100	+51,100 +44,000	+49,900 +46,800	+50,900	
102		−1,425		−600	+1,500	+1,875	+2,250	+1,275							
104				−225	−5,300 −7,800	−2,750 −4,600	−3,800	−5,775					−7,700		
106				+225	−4,700 −6,400	−3,000 −3,400									
108				+300	−3,600 −5,000	−2,600 −2,600									
110				+375	−3,700 −5,500	−1,500 −1,900									
112				+1,125	−3,400	+375 +1,700									
114				+1,200	−1,200 −2,800	+1,100 −800									

+ = Tension
− = Compression.

TABLE XVI.—REINFORCEMENT STRESSES, BARR BUILDING TEST PANEL.—*Continued.*

Gauge Point.	Unit Load.	75.5	150	150	300	380	380	500	527	550	567	591	615	650	650
	Stage.	2	4	4	11	13	13	14	15	16	17	18	19	20	20
	Date and Hour.	Nov. 25 4 A. M.	Nov. 28 10 A. M.	Dec. 2 12 M.	Dec. 2 3 P. M.	Dec. 3 10 A. M.	Dec. 5 4.30 P. M.	Dec. 5 11 A. M.	Dec. 6 12.30 P. M.	Dec. 6 2 P. M.	Dec. 6 2 P. M.	Dec. 6 3 P. M.	Dec. 7 10 A. M.	Dec. 7 3.30 P. M.	Dec. 8 9.30 A. M.
116				+3,450	+980 −1,600	+3,400 +2,100									
118				+3,525	+2,000 −1,100	+4,800 +4,400									
120				+2,475	+2,300 +300	+6,300 +2,300									
122		−2,100	+4,500	+3,675	+10,900 +9,500	+19,700 +30,000	+19,500	+30,000	+33,800	+36,750	+36,900	+40,500 +39,800	+39,200 +45,500	+72,400	+115,000
124		+1,050	+8,600	+7,125	+14,600 +13,500	+21,700 +21,900	+21,000	+28,200	+31,400			+37,200	+42,800	+46,800	+48,800
126		+1,200	+8,700	+7,650	+14,700 +10,000	+22,400 +19,300	+22,000	+32,900	+35,650			 +22,700	+53,000 +47,700	+58,300	+61,500
115		−2,100		−2,400	−2,600	−900	−2,400	−3,450							
117				−3,000	−5,800 −6,000	−6,900 −6,400	−7,130	−9,300				 −10,200	 10,100		
119			+3,750	−2,555	−4,900 −5,500	−5,500 −5,900						 −7,500	 −7,600		
121			+7,400	−900	−3,500 −4,100	−2,850 −2,900						 −3,700	 −2,700		
123				−450	−2,400 −2,500	−1,050 −375						 +1,100	 +2,800		

+ = Tension.
— = Compression.

TABLE XVI.—REINFORCEMENT STRESSES, BARR BUILDING TEST PANEL.—*Continued.*

Gauge Point.	Unit Load.	75.5	150	150	300	380	380	500	527	550	567	591	615	650	650
	Stage.	2	4	4	11	13	13	14	15	16	17	18	19	20	20
	Date and Hour.	Nov. 25 4 A. M.	Nov. 28 10 A. M.	Dec. 2 12 M.	Dec. 2 3 P. M.	Dec. 3 10 A. M.	Dec. 5 4.30 P. M.	Dec. 5 11 A. M.	Dec. 6 12.30 P. M.	Dec. 6 2 P. M.	Dec. 6 2 P. M.	Dec. 6 3 P. M.	Dec. 7 10 A.M.	Dec. 7 3.30 P. M.	Dec. 8 9.30 A. M.
125	......	......	......	+450	−820	+825	......	......	......	......	......	......	......	......	......
	......	......	......	......	−1,700	−230	......	......	......	......	......	+6,400	+9,900	......	......
127	......	......	......	+2,025	+1,050	−3,400	......	......	......	......	......	......	......	......	......
	......	......	......	......	+600	+2,500	......	......	......	......	......	+11,800	+14,900	......	......
129	......	......	......	+2,550	+3,400	+6,000	......	......	......	......	......	......	......	......	......
	......	......	......	......	+1,900	+4,300	......	......	......	......	......	+14,800	+19,200	......	......
131	......	......	......	+7,125	+3,800	+6,400	......	......	......	......	......	......	......	......	......
	......	......	......	......	+2,200	+6,900	......	......	......	......	......	+16,300	+19,800	......	......
133	......	......	......	+8,800	+12,700	+20,600	......	......	......	......	......	......	......	......	......
	......	......	......	......	+3,300	+8,700	......	......	......	......	......	......	......	......	......
135	......	......	......	+7,275	+9,300	+16,100	......	......	......	......	......	......	......	......	......
	......	......	......	......	−70	+5,400	......	......	......	......	......	......	......	......	......
137	......	......	......	+2,775	+2,000	+8,600	......	......	......	......	......	......	......	......	......
	......	......	......	......	−75	+5,500	......	......	......	......	......	......	......	......	......
139	......	......	......	+4,725	−3,200	+6,700	......	......	......	......	......	......	......	......	......
	......	......	......	......	+1,300	+2,500	......	......	......	......	......	......	......	......	......
141	......	......	......	+3,975	+40	+4,000	......	......	......	......	......	......	......	......	......
	......	......	......	......	+2,500	−600	......	......	......	......	......	......	......	......	......
143	......	......	......	+2,550	−45	+1,350	......	......	......	......	......	......	......	......	......
	......	......	......	......	+2,400	+300	......	......	......	......	......	......	......	......	......
145	......	......	......	+2,175	−900	−225	......	......	......	......	......	......	......	......	......
	......	......	......	......	−4,000	−2,200	......	......	......	......	......	......	......	......	......

+ = Tension.
− = Compression.

TABLE XVI.—REINFORCEMENT STRESSES, BARR BUILDING TEST PANEL.—*Continued.*

Gauge Point.	Unit Load.	75.5	150	150	300	380	380	500	527	550	567	591	615	650	650
	Stage.	2	4	4	11	13	13	14	15	16	17	18	19	20	20
	Date and Hour.	Nov. 25 4 A. M.	Nov. 28 10 A. M.	Dec. 2 12 M.	Dec. 2 3 P. M.	Dec. 3 10 A. M.	Dec. 5 4.30 P. M.	Dec. 5 11 A. M.	Dec. 6 12.30 P. M.	Dec. 6 2 P. M.	Dec. 6 2 P. M.	Dec. 6 3 P. M.	Dec. 7 10 A. M.	Dec. 7 3.30 P. M.	Dec. 8 9.30 A. M.
147	......	−75	....	+750	−3,200 −4,000	−2,500 −3,900	−2,550	−4,050	....	....	....	....	....	....	....
149	......	....	....	....	−3,300	−2,550	−2,930	−4,050	....	....	....	....	....	....	....
128	......	+675	....	+3,750	+7,100	+9,750	+9,000	+12,750	+13,500	....	....	....	+19,150	+21,750	+22,600

+ = Tension.
− = Compression.

Fig. 66 shows the location of gauge points at which reinforcement stresses were read. A summary of the observed stresses is given in Table XVI, together with the load and time at which each set was taken. The two figures given for each point are by the two different observers. Figs. 67 and 68 show load-stress

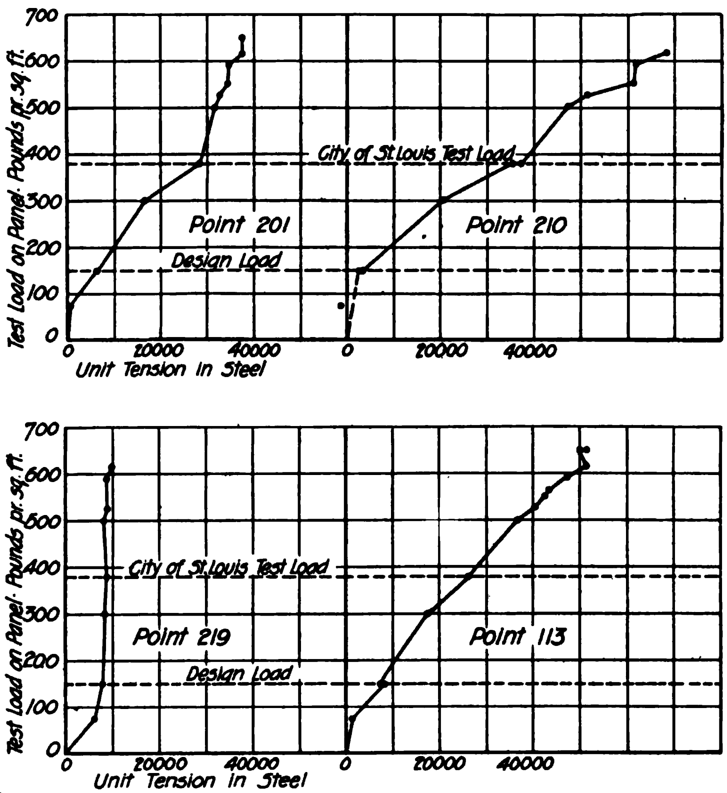


FIG. 67.—DIAGRAM OF STRESSES IN REINFORCEMENT.

curves for a few of the bars at critical points and for the structural steel supporting beams.

Fig. 69 shows the distribution of stress in the bars which are on the short span. The section shown is taken along the long axis of the panel and the stresses plotted are those in the bars cut by the section. Similar curves are shown in Fig. 70 for the

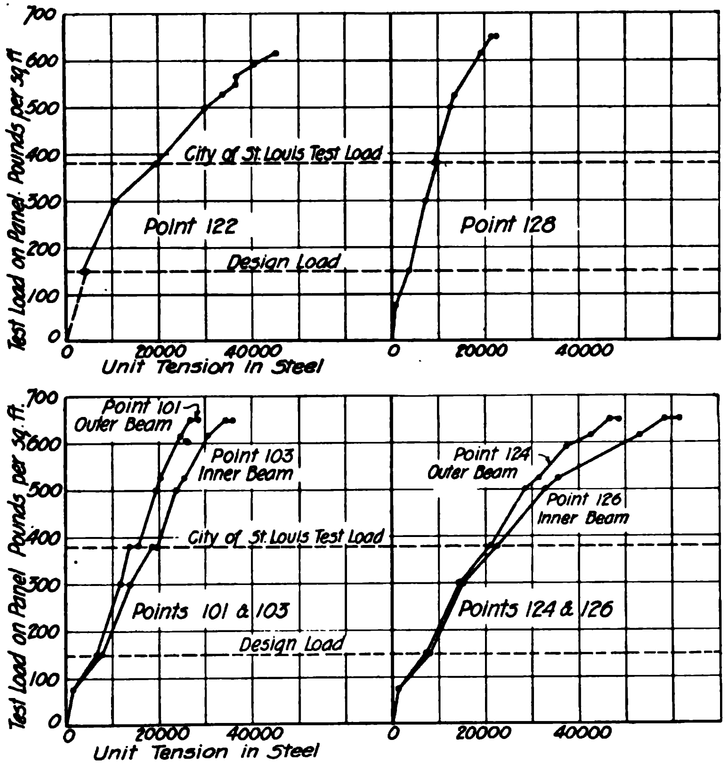


FIG. 68.—DIAGRAM OF STRESSES IN REINFORCEMENT AND IN STEEL SUPPORTING BEAMS.

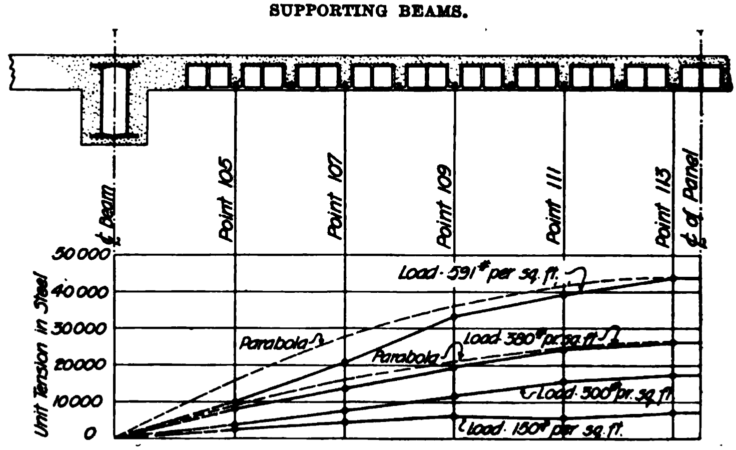


FIG. 69.—DISTRIBUTION OF STRESS IN THE REINFORCEMENT CARRYING THE LOAD ACROSS THE SHORT SPAN.

top bars over the supporting beams. Some of these curves may be distorted because the supporting beams tipped up on the bearing plates and spread apart at the corners, thus inducing tension across the diagonals of the slab.

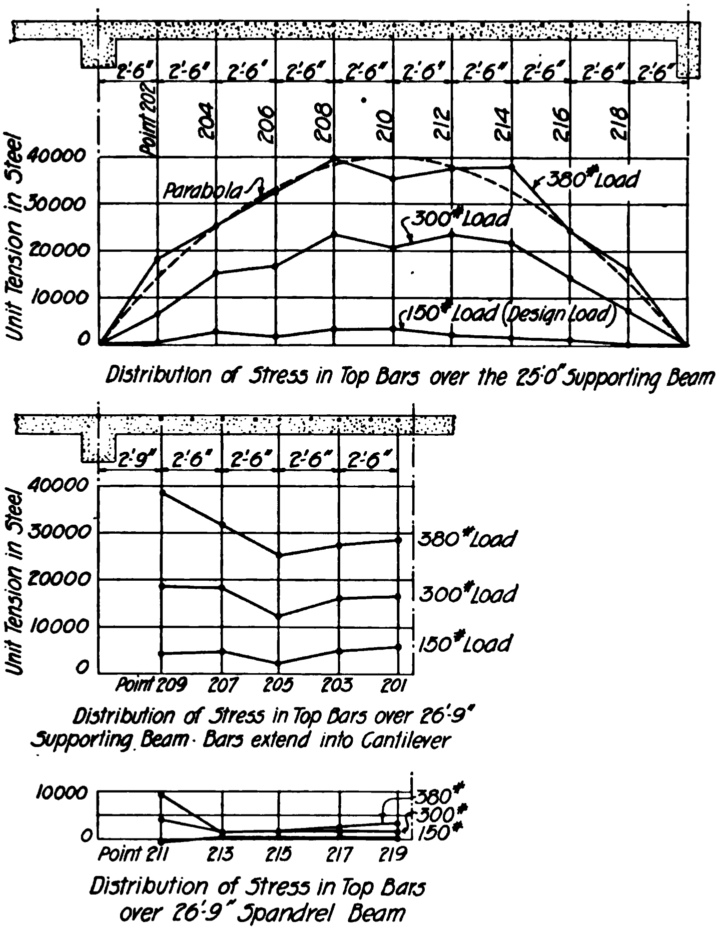


FIG. 70.—DIAGRAMS SHOWING DISTRIBUTION OF REINFORCEMENT STRESS.

At the 500-lb. load, cracks could be found extending a short distance from the corners of the panel along the diagonals. Fig. 71 shows the location of cracks at a load of 650 lb. per sq. ft.

In order to locate the points of inflection and to determine the bond stresses, readings were taken along portions of three bottom bars. The results of these readings are plotted in Figs. 72 and 73.

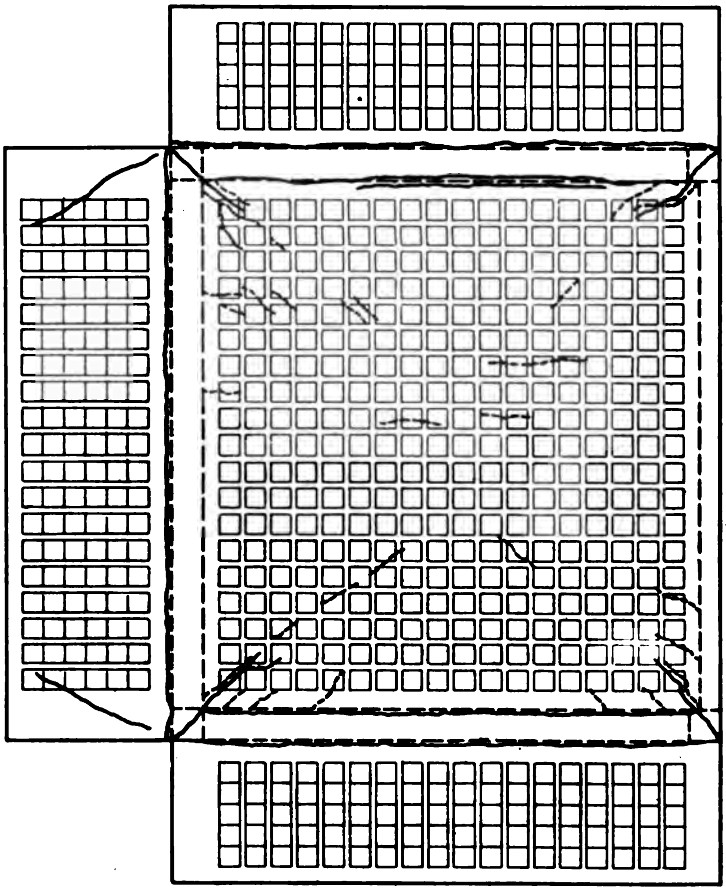


FIG. 71.—PLAN SHOWING LOCATION OF CRACKS.

A summary of the stresses and deflections at the critical points of the slab is given in Table XVII.

No definite conclusions as to the distribution of stress among

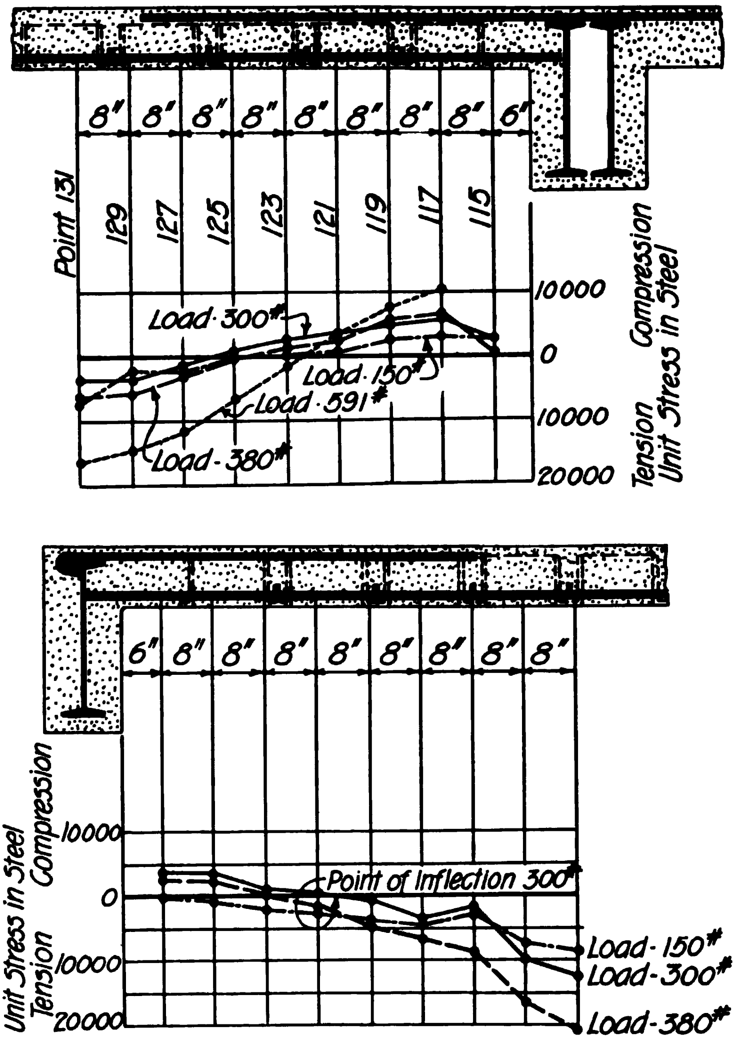


FIG. 72.—VARIATION OF STRESS AND LOCATION OF POINT OF INFLECTION ALONG MIDDLE BAR IN DIRECTION OF SHORT SPAN.

TABLE XVII.—SUMMARY OF PRINCIPAL STRESSES AND DEFLECTIONS, BARR BUILDING TEST PANEL.

DEFLECTIONS AT CENTER.

Design Load 150 lb. per sq. ft.	0.167 in., or $\frac{1}{1670}$ of Span
St. Louis Building Code 380 lb. per sq. ft.	0.860 in., or $\frac{1}{325}$ of Span

CONCRETE STRESSES.

Location.			Gauge Point.	Design Load 150 lb. per sq. ft.			St. Louis Code 380 lb. per sq. ft.		
				L. L.	D. L.	Total.	L. L.	D. L.	Total.
Top of Slab at Center	Long Span		402	100	100	200	675	100	775
	Short Span		401	100	100	200	800	100	900
Bottom of Slab at Flange of Supporting Beam	Long Span		304	325	150	475	1,250	150	1,400
	Short Span	Fixed End	301	400	200	600	1,525	200	1,725
		Free End	309	150	75	225	425	75	500
Bottom of Slab on Rib just outside of Beam Flange	Long Span		308	175	75	250	925	75	1,000
	Short Span	Fixed End	303	400	200	600	1,575	200	1,775
		Free End	311	175	100	275	700	100	800

REINFORCEMENT AND STEEL STRESSES.

Location.			Gauge Point.	Design Load 150 lb. per sq. ft.			St. Louis Code 380 lb. per sq. ft.		
				L. L.	D. L.	Total.	L. L.	D. L.	Total.
Bottom of Slab at Center	Long Span		122	3,500	2,000	5,500	19,500	2,000	21,500
	Short Span		113	7,500	3,500	11,000	26,500	3,500	30,000
Top of Slab at Support	Long Span		210	3,500	1,500	5,000	37,500	1,500	39,000
	Short Span	Fixed End	201	6,500	3,500	10,000	28,500	3,500	32,000
		Free End	219	7,500	4,000	11,500	8,500	4,000	12,500
·	Short Span		101 & 103	7,000	3,500	10,500	17,500	3,500	21,000
Bottom Flange of I Beam	Long Span	Spandrel Beam	128	3,500	2,000	5,500	9,500	2,000	11,500
		Interior Beam	124 & 126	7,500	4,000	11,500	22,000	4,000	26,000

NOTE.—Stresses due to dead load are obtained by projecting the stress curve to the zero load.

the two sets of ribs can be reached from the data obtained in this one test, although the stresses obtained indicate that about

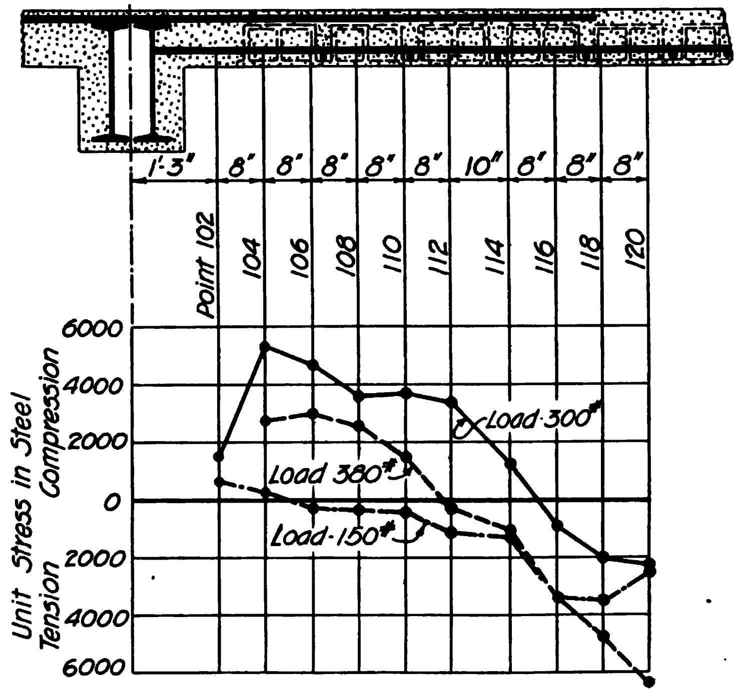


FIG. 73.—VARIATION OF STRESS AND LOCATION OF POINT OF INFLECTION ALONG MIDDLE BAR IN DIRECTION OF LONG SPAN.

65 per cent of the load was carried across the short span of the panel, instead of the 55 per cent assumed.

DISCUSSION.

Mr. Ash. MR. L. R. ASH.—I would like to know if experiments have been made to determine whether the modulus of elasticity of high carbon steel is really more than that of medium steel. Some claim that high carbon steel should be used because of the increased modulus of elasticity.

One other question is whether experiments have been made to determine the distribution of stresses in a continuous slab with a concentrated load. A problem that frequently comes to the bridge designer is how to take care of a continuous slab which has large dimensions one way and small dimensions the other, for example a slab supported between two continuous stringers.

Mr. Talbot. MR. ARTHUR N. TALBOT.—The modulus of elasticity of steel is a fairly definite property. It varies from 29,000,000 lb. per sq. in. to 31,000,000, perhaps; the average of tests would bring it a little under 30,000,000 lb. per sq. in. I do not know that anything has been found which explains or gives the causes of this slight variation. I am quite sure, however, from tests which we have made, that there is no appreciable difference in the modulus of elasticity between the high carbon and the mild steel.

It is an important question to determine to what extent it is necessary to reinforce a slab laterally when a concentrated load is carried. We have been making tests along this line at the laboratory of the University of Illinois for three years and we hope to have results to give out soon. I may say now that I have been surprised to find how little reinforcement is necessary laterally in distributing the stresses sidewise. In these tests we have used beams 30 and 48 in. and longer, with depths of 4 to 7 in. and with widths of 30 to 48 in. These slabs were loaded in several ways, by applying the load entirely across the beam, by applying it over half the width of the beam and also over 1/5 of the width and over 1/10 the width. In a general way it may be said that with as much as $\frac{1}{4}$ of one per cent of lateral reinforcement the stresses are distributed laterally over a width of say 8 or 10 times the depth of the beam.

MR. W. P. ANDERSON.—In making the comparison with the bending moments, what distance was used for the span. Mr. Anderson.

MR. TALBOT.—A definite length had to be assumed. It was thought best to use something more than the clear span and 3 in. more than the clear span for both girders and beams was used. This is somewhat shorter than that given by the usual methods of design. Mr. Talbot.

MR. W. K. HATT* (By Letter).—The writer has read this important contribution with a great deal of interest. Mr. Hatt.

It is realized that our methods of computing the strength of a continuous construction of slabs, floor beams, and columns of a reinforced concrete building is conventional and very largely empirical. We need many such tests as are described in this report to determine the allowable limits of deflection and the real factors of safety obtaining in the usual designs. The common practice of loading one panel is evidently misleading since surrounding panels under load assist in carrying the load.

The report confirms the opinion many have had that under ordinary working loads the tensional stresses in the concrete assist the reinforcement in carrying the bending moment and that the conventional computal stress of 16,000 lb. will not be found in the test floor. Of course under this distribution the compressional stresses will be the critical stresses.

The ratios between the compressional stresses and the computed reinforcement stresses in concrete beams reinforced with various percentages of metal is an important factor in the work of the designer, who is controlled by building laws. This report does not comment on the proper values of this ratio. The indication from the tests, however, is that in spite of high compressional stresses failures in compression are not evident. The recent publication of the Bureau of Standards by Messrs. Richard L. Humphrey and Louis Losse, fixes an extreme fiber stress in compression of 1000 lb. per sq. in. at the measured unit stress of 16,000 lb. per sq. in. in the reinforcement for 1–2–4 concrete.

The writer made a careful test of the Franks building in Chicago in the summer of 1911 and submits report of this test as a contribution to the discussion of this subject.

* Professor of Civil Engineering, Purdue University, Lafayette, Ind.

REPORT ON TEST FOR ACTUAL STRESSES OF THE A. J. FRANKS BUILDING, CHICAGO, ILLINOIS.

BY W. K. HATT, CONSULTING ENGINEER, PROFESSOR OF CIVIL ENGINEERING, PURDUE UNIVERSITY.

During the latter part of August, 1911, the undersigned loaded panels of a reinforced concrete building, Fig. 1, constructed by the Leonard Construction Company, for Mr. A. J. Franks, and measured the actual deformations of the concrete and steel under load. The test was performed upon the basis of specifications prepared by the undersigned.

In brief, the panels were loaded with pig iron in increments, and the accompanying deformations were measured in the steel and in the concrete at all critical points, with a view to fixing a safe limit of loading and to understanding the mechanical action of the structure.

TEST STRUCTURE.

The building tested is a ten-story and basement warehouse intended for the printing and paper trades, Dwight Bros. Paper Company, tenants. The type of construction used was the Cantilever Flat Slab System, the reinforced concrete setting drawings and shop details being made by the Concrete Steel Products Company, Engineers, of Chicago. The architects of the building were Richard E. Schmidt, Garden and Martin. The design load on the floors was taken at 250 lb. per sq. ft. The panel dimensions were 19 ft. 4 in. by 20 ft. 3 in. The four panels under observation were in the interior of the building and on the tenth floor where the columns were of minimum size. As the effective clear span between capital and the eccentric action on the columns were the greatest, the location was such as to insure the most severe test possible.

OBSERVERS AND METHODS OF OBSERVATION.

The observations were made by experienced observers as follows: Professor H. H. Scofield, of Purdue University; Professor W. A. Slater, of the University of Illinois Engineering Experiment Station; Mr. W. E. Ensign, of the University of Illinois Engineering Experiment Station; with the assistance of Professor L. W. Weeks, of Purdue University.

The deformations were determined by the use of extensometers of the type devised by Professor H. C. Berry of the University of Pennsylvania. On the steel a gauge length of 10 in. was used, which, with the multiplying lever in the instrument, gave direct readings of unit deformations of .00002 in. per in., corresponding to stresses of 600 lb. per sq. in. in the steel, and it was possible to estimate clearly fractions of this amount. On the concrete readings the gauge length was 6 in. and the direct reading of unit deformations was .000033, corresponding to a stress of 133 lb. per sq. in., and it was possible to estimate fractions of this amount with accuracy.

Errors in operating the instrument were reduced to a minimum by taking every reading at least five times and by calibrating on a standard bar at

frequent intervals. Four standard bars were used throughout the test, all of these being embedded in the concrete and subject to the same temperature changes as the rods in the test floor but free from any stress due to applied load. By reading on these bars between sets of five or six test readings the

FIG. 1.—A. J. FRANKS BUILDING IN COURSE OF CONSTRUCTION, CHICAGO, ILL.

observations on the materials under test were freed from temperature differences and systematic errors.

The deflections were measured to .0001 in. by use of the deflectometer described in a previous test for actual stresses.

Tensile deformations were measured throughout the test over 42 gauge lengths on the steel reinforcing rods; compressive deformations in the concrete were measured over 26 gauge lengths; deflections were observed at 24 points, and 27 other readings of deformations were taken throughout the test to study such phenomena as the arch and slab action, the distribution of

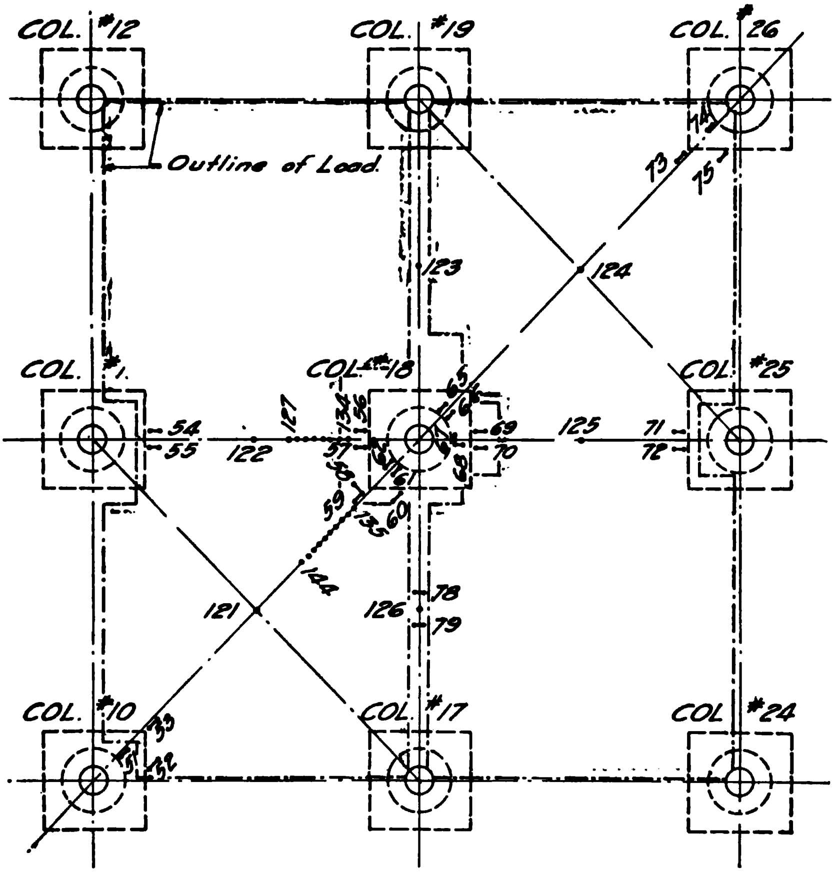


FIG. 2.—LOCATION OF GAUGE LENGTHS IN CONCRETE SLAB.

stress, and the eccentric loading on the columns at the edges of the loaded area. The location of the gauge lengths is given in Figs. 2 and 3.

The observations were arranged in groups, each designed to cover adequately some particular feature, and symmetrically located observations were obtained as a check in every case where possible, in order to cover variations in the quality of the concrete at different points.

The conditions were exceptionally favorable to a satisfactory test. The building was practically completed at the time of test and the variation in temperature was very slight, being about 74° at the start of the test, dropping gradually and uniformly to 70°, and rising again at the end to 74°.

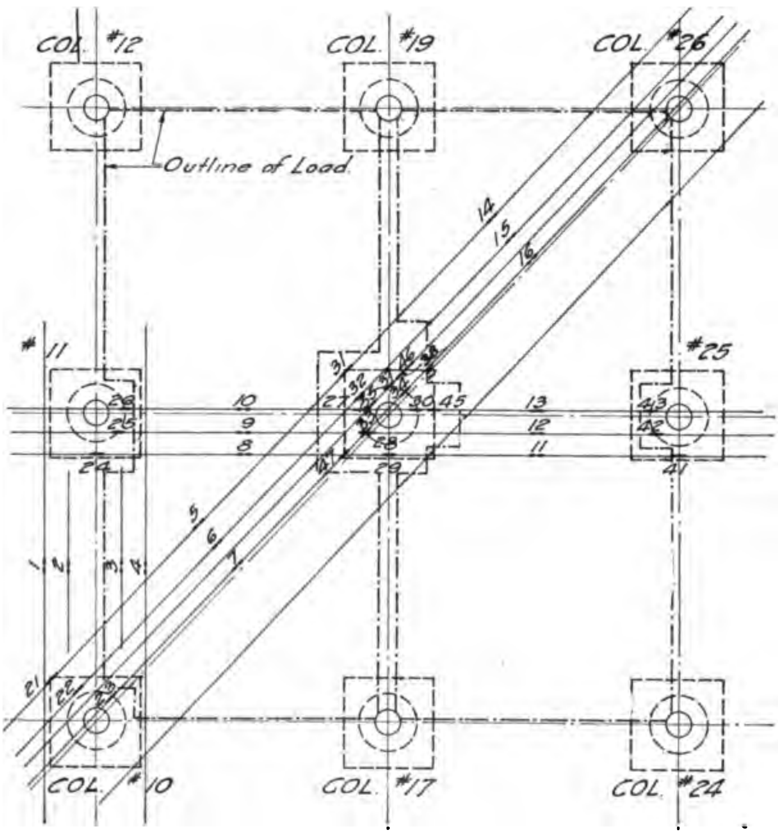


FIG. 3.—LOCATION OF GAUGE LENGTHS IN REINFORCING RODS.

The slab was poured on June 23, 1911, and was 64 days old when the maximum load of 624 lb. per sq. ft. of panel area was placed upon it.

LOADING.

The amount of loading was determined by the weight of the pig iron as recorded on the weigh bills delivered by the teamsters, and was checked by weighing a number of piles of pig iron from the test load on a platform scales.

The pig iron was piled on the floor in separate piers, Fig. 4, each placed

FIG. 4.—FOUR PANELS PARTIALLY LOADED, FRANKS BUILDING.

within a two-foot square, so that no arch action existed in the load itself to relieve the panel from moment. It was necessary to leave several of these squares vacant about columns and to leave an aisle between the columns to allow space for observations. The pig iron belonging on these vacant squares was distributed over the remaining squares of the same panel. It will be apparent that under this procedure the real load intensity, as affecting bending stresses and deflections, is the intensity of loading over the loaded area (about 90 per cent of panel area), rather than the nominal or average load over the entire panel area. It is evident that had the 10 per cent of panel area close to the columns been loaded to the same intensity the increase in stresses and deflection would have been practically nil.

All gauge lengths were measured and checked throughout before loading was started. When the loading reached certain amounts, distributed evenly over four panels, it was discontinued and allowed to stand for 6 hours before the observations were made and the loading resumed. The increments of load at which measurements were made were as follows: 75, 150, 256 (the design load), 312 (intensity, 359), and 624 (intensity, 717) lb. per sq. ft., the latter load being applied to two panels only. Readings were also taken with 256 on two diagonal panels and 312 on the other two panels, and with 468 on two diagonal panels and 156 on the others, but without waiting for the six-hour interval to elapse before taking readings. The total number of complete observations over single gauge lengths was over 2,000, and the total individual readings over 10,000.

STRESSES.

The stresses were determined from the observed deformations by using a modulus of elasticity of 30,000,000 lb. per sq. in. for the steel, and 4,000,000 lb. per sq. in. for the concrete. The latter value was determined from tests of three concrete prisms poured from the concrete in the test slab and tested at Purdue University at an age of 77 days.

Table I gives a summary of the corrected values of the total dead and live load stresses observed in the various groups of observations. The detailed summary of individual stresses at the various observation points are omitted in this report. After the observations had been corrected for temperature and observational errors, by use of the standard bar calibrations, load deformation curves were plotted for each observation point, the known nature of the load deformation curve under flexure being used as a basis. The dead load stress has been taken from these curves as equal to the stress caused by an equal live load.

COMMENTS ON THE RESULTS.

The undersigned is not prepared at the present time to state the significance of the results obtained with respect to the mechanics of this form of construction. Such statements would have somewhat of a speculative element and should be separated from the report of test which is one of measured facts.

As compared with the design requirements of the Chicago Building Code, it is interesting to note that at design load the highest average stress in the steel is 5078 lb. per sq. in., while the highest average compressive

TABLE I.—CONDENSED SUMMARY OF ACTUAL STRESSES.

All stresses are the final values for total dead and live load, and are given in lb. per sq. in.

Average of Observation Nos.	Description.	Live Load—lb. per sq. ft. 236	312	624
	Stresses in Slab Rods.			
5, 6, 7, 14, 15, 16	Center of span, diagonal band	1071	1920	6950
8, 9, 10, 11, 12, 13	Center of span, cross band	4539	6140	10095
31, 32, 33, 34, 35, 37	Over capital at center column, diagonal band	3440	4350	9280
27, 28, 29, 30	Over capital at center column, cross band	4575	4840	8140
21, 22, 23	Over capital at corner column, diagonal band	1920	2280	7540
24, 25, 26, 41, 42, 43	Over capital at side column, cross band	2690	3138	5315
	Compressive Stresses in Concrete.			
56, 57, 59, 60, 68, 69, 70	On slab at center column	560	650	1206
61, 62, 65, 66	On drop at center column	677	778	1685
51, 74	On drop at corner column	318	370	1515
52, 53, 73	On slab at corner column	329	378	650
54, 55, 71, 72	On slab at side columns	189	217	420
	Maximum Stresses in Columns Due to Eccentric Live Load.			
104	Compression in concrete, corner column	680	840	1660
109	Compression in concrete, side column	416	512	1000
84	Tension in steel, corner column	4980	6000	11620
99	Tension in steel, side column	2220	2640	5880
	Deflections in Inches.			
121, 124	Center of panel—at 6 hours	.123	.156	.475
	Center of panel—at 24 hours	...	...	.500
	After standing unloaded 6 hours	.142		
	After standing unloaded 2½ days	.090		

stress in the concrete is 677 lb. per sq. in. On the basis of safe working stress in the steel of 16,000 lb. per sq. in., and in the concrete of 35 per cent of the ultimate strength (which averaged over 3250 by tests of prisms) or 1100 lb. per sq. in.; it appears that the steel is stressed to 31 per cent

of its safe load while the concrete is stressed to 62 per cent of its safe load. It appears, therefore, that the design is overbalanced with an excess of steel. At the highest applied load of 717 lb. per sq. ft., the ratio of the steel and concrete stresses remains practically unchanged.

The eccentric action of the test load was most marked on the corner columns of the loaded area and was sufficient to produce a tension in the steel of 5000 lb. per sq. in.

With respect to the strength of the structure, it may be said that a nominal load of 624 lb. per sq. ft. of panel, actually 717 lb. per sq. ft. of loaded surface, was applied without producing any permanent damage to the building. At this load the highest observed average total dead and live load stresses were less than 12,000 lb. per sq. in. in the steel and less than 1700 lb. per sq. in. on the concrete.

From a consideration of the data from the above test the writer concludes that the A. J. Franks Building is amply strong to carry the designed load and that the lower floors at least may safely and continuously be loaded with considerably more than the designed load.

(Signed) W. K. HATT.

THE TESTING OF REINFORCED CONCRETE BUILDINGS UNDER LOAD.

By W. A. Slater.*

I. Introduction.

Development of Building Tests.—For several years there has been a growing demand for tests of full-size structural members. A more recent development is the test of structures themselves and the measurement of actual stresses in the component parts.

Load tests have been required by city building departments as a condition of acceptance of reinforced concrete buildings and have been used by construction companies and engineers to demonstrate the adequacy of various designs. Such load tests are never continued to destruction, the applied load being generally twice the design live load, and emphasis is placed upon measurement of deflection and recovery. No measurements of stresses are made in such tests and under these conditions the safe load can not be fixed upon as a definite ratio of the ultimate load. The deflections observed in such tests constitute a very inadequate and actually misleading measure of the stresses. Slight deflections have been taken to indicate low stresses in reinforcement and in concrete, but recent tests in which deformations were measured have shown that even with slight deflections large stresses are developed in concrete even when the reinforcement stresses are low. The tendency of building codes was to disregard continuity of action in beams in reinforced concrete buildings and to specify the design as of simple beams, but even in such cases a small amount of reinforcement was placed across the support to prevent the opening of large cracks. This reinforcement and the tensile strength of the concrete have been sufficient to develop a large stress in the concrete at the support which may not have been specifically provided for. Thus the so-called conservative attitude of not allowing anything for continuity of beams at the support may prove a source of weak-

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ness. The measurements of deformations in building structures confirms the truth of this statement.

Purpose and Scope of this Paper.—The reports of all building tests in which deformations have been measured deal in the main with the behavior of the structure and record the results, and are not primarily concerned with the working of the instruments or with the methods of making the tests. To conduct a successful building test is difficult, however, and this paper is written in order to present information as to methods of testing gained by experience and to point out certain respects in which such tests

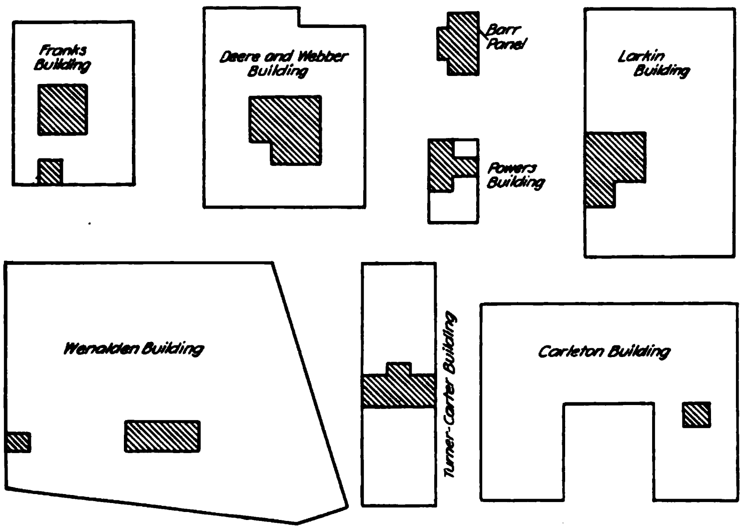

FIG. 1.—RELATIVE SIZE OF FLOOR AREAS TESTED.

may be conducted more satisfactorily than those which have already been made. The following general order of presenting the material in hand will be observed: (1) enumeration of tests, (2) the planning and preparation for a test, (3) the instruments; their construction and use, and the methods of making observations, (4) the methods of making calculations and (5) the cost of a test.

The following is a list of tests of building floors in which the methods described herein of measuring deformation were used. Fig. 1 shows the range in size of these test areas.

Test No. 1.—Deere and Webber Building, Minneapolis, Minnesota, October and November, 1910; flat slab floor with four-way reinforcement; built by Leonard Construction Company of Chicago, and tested by them with the co-operation of the Engineering Experiment Station of the University of Illinois.

Test No. 2.—Wenalden Building, Chicago, Illinois, June and July, 1911. Beam and girder building constructed by Ferro-Concrete Construction Company of Cincinnati, and tests made by co-operation between the National Association of Cement Users, the Ferro-Concrete Construction Company and the Engineering Experiment Station of the University of Illinois.

Test No. 3.—The Powers Building, Minneapolis, Minnesota, July and August, 1911; flat slab floor with two-way reinforcement; built and tested by Corrugated Bar Company of St. Louis.

Test No. 4.—Franks Building, Chicago, Illinois, August, 1911; flat slab floor with four-way reinforcement; built and tested by Leonard Construction Company of Chicago.

Test No. 5.—Turner-Carter Building, Brooklyn, New York, September, 1911; beam and girder floor; built by Turner Construction Company of New York; test made by co-operation between National Association of Cement Users, the Turner Construction Company and the Engineering Experiment Station of the University of Illinois.

Test No. 6.—Carleton Building, St. Louis, Missouri, October, 1911; flat slab floor with two-way reinforcement; built and tested by Corrugated Bar Company.

Test No. 7.—Barr Building, St. Louis, Missouri, December, 1911; full size test panel (25 ft. x 26 ft. 9 in.). Terra-cotta tile used to lighten construction; gives two-way T-beams with web between tile on tension side and concrete flange above the tile; two-way reinforcement. Panel built by Corrugated Bar Company to demonstrate efficiency of design proposed for Barr Building in St. Louis; test made by Corrugated Bar Company.

Test No. 8.—Ford Motor Building, Detroit, Michigan, February and March, 1912; flat slab floor; built and tested by the Corrugated Bar Company.

These seem to be the only full-size reinforced concrete floor tests on record in which deformations in reinforcement and concrete have been measured. The writer was in immediate charge

of Tests Nos. 2 and 5 and had a part in the conduct of all the others except Tests Nos. 6 and 8. The methods of testing presented in this paper were developed by the writer as a result of his connection with the tests. These methods were designed to increase the accuracy of results, to avoid accidental errors and to correct for systematic errors.

Much credit for the initiative in this type of test is due Mr. A. R. Lord, formerly research fellow at the University of Illinois, who was largely instrumental in bringing about the test of the Deere and Webber Building, the first in the series named. After the presentation of Mr. Lord's paper on the test of the Deere and Webber Building, The National Association of Cement Users decided to continue the investigation. All of the tests given in the above list were conducted on the same general lines as that of the Deere and Webber Building. Only the tests of the Wenalden Building and the Turner-Carter Building were in the series authorized by the National Association of Cement Users, but the results of the tests made by the Corrugated Bar Company on the Powers Building and on the Barr Building test panel have been placed at the disposal of the Association. The Franks Building test, made by the Leonard Construction Company, was an investigation planned to give data for an intelligent modification of the Chicago Building Code. The other two tests, those of the Carleton Building and the Ford Motor Building, were in the nature of investigation of special features of design. The methods used in all of these tests are essentially the same and have been developed at the University of Illinois Engineering Experiment Station.

Available Literature.—Reports of results of some of these tests are available as follows:

1. Deere and Webber Building.
   Paper by A. R. Lord, "A Test of a Flat Slab Floor in a Reinforced Concrete Building." *Proceedings*, Vol. VII, 1911.
   Abstracts: *Engineering News*, December 22, 1910; *Engineering-Contracting*, December 22, 1910.
2. Wenalden Building and Turner-Carter Building.
   Report of Committee on Reinforced Concrete and Building Laws, Part I. *Proceedings*, Vol. VIII, 1912.*

* See pp. 61–167.—ED.

Abstracts: *Engineering Record*, March 23 and April 13, 1912; *Engineering News*, April 18, 1912; and *Cement Age*, April, 1912.

3. Powers Building and Barr Building Test Panel.
   Report of Committee on Reinforced Concrete and Building Laws, Parts II and III. *Proceedings*, Vol. VIII, 1912.*
   Abstracts: *Engineering News*, April 18, 1912; and *Engineering Record*, April 20, 1912.
4. Franks Building.
   (*a*) Abstracts of paper by W. K. Hatt† before Indiana Engineering Society, *Engineering-Contracting*, March 13, 1912; and *Engineering News*, April 8, 1912.
   (*b*) Discussion by W. K. Hatt on Report of Committee on Reinforced Concrete and Building Laws, *Proceedings*, Vol. VIII, 1912.†
   (*c*) Trade publication on Cantilever Slabs published by Concrete Steel Products Company, Chicago, Ill.

II. CONDUCT OF TESTS.

DEFINITIONS.

In the following descriptions of tests, many terms will be used for which somewhat arbitrary definitions will need to be made. These definitions are:

Gauge Hole: A small hole (.055 in. is here recommended) drilled into the steel bar or into the plug inserted in the concrete has been termed a gauge hole. It is for the admission of the point of a leg of the extensometer.

Gauge Line: The gauged length connecting a pair of gauge holes is termed a gauge line.

Reading: A reading is a single observation on any gauge line.

Observation: An observation as here used is the average of a number of readings.

Zero Length of Instrument: The length of the instrument at the time of taking the first observation on the standard bar will be known as the zero length of the instrument. This first observa-

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* See pp. 61–157.—ED.

† See pp. 159–167.—ED.

tion on the standard bar is not the zero length, but a comparison of a subsequent observation with it shows any change from the zero length.

Correction: A correction is the amount which if added algebraically to the observation will give the observation which would have been obtained if the instrument had not changed from its zero length.

Series of Observations: The observations taken consecutively at a given load without repetitions on any gauge line are defined as a series of observations.

Interval: An interval as used here is the time elapsing between consecutive observations, and all intervals in any series are (for lack of more exact information) assumed to be equal. For this purpose the average of two consecutive observations on standard gauge lines is considered a single observation.

Standard Gauge Line: This is a gauge line used usually to determine changes of length of instrument, of reinforcement or of concrete due to other causes than the applied load. Its purpose usually is to determine the temperature effect on the instrument, but it may be used to detect accidental changes of instrument or temperature stresses in the reinforcement or the concrete. Originally this gauge line was placed on a steel bar separate from the structure, and this gave rise to the term standard bar. In several of the later tests, however, the standards have consisted of gauge lines placed in the reinforcement and concrete of the structure remote from the area affected by the load. Standard gauge line is adopted, therefore, as the more general term and any reference to the standard bar may be understood to signify the standard gauge line on a bar separate from the structure.

GENERAL OUTLINE OF METHOD OF TESTING.

After determining what measurements will best give the information desired from the test, the gauge lines are laid off on the surface of the concrete and small holes are cut or drilled in the concrete at a predetermined distance apart in order to expose the reinforcement or allow a metal plug to be inserted, according as the measurement is of reinforcement or concrete deformation. The metal plugs used are securely held in place by imbedment

in plaster of Paris. The gauge holes having been carefully prepared, a set of zero readings is taken on all gauge lines, an increment of the loading material is then added and a second series of observations on the gauge lines taken. The difference between the two readings on the same gauge line represents the deformation in that gauge line. It is possible that this apparent deformation may be due partly to temperature changes in the instrument instead of stress changes of the material by reason of applied load. For this reason reference measurements are made on standard unstressed bars made of Invar steel which has a very low coefficient of expansion and whose change in length due to temperature would therefore be very slight. From these readings on the standard bar, temperature corrections are computed as shown in a later paragraph and applied to the observations in order to determine the actual change in length of the gauge line. Another increment of load is then applied and another series of observations taken.

Floor deflections also have been measured in all of these tests, but they have been considered as of secondary importance. They have been used to throw light on the correctness or incorrectness of the deformation readings and to gain some idea of the general distribution of stresses throughout a floor. They can apparently be depended upon to show with considerable accuracy the proportional rate of increase of stress, but deflection formulas are so imperfect that measurement of deflections can not be depended upon to give the actual values of stresses.

Measurements of dimensions such as span, depth of beams, location of observation points, weight of loading material, location of cracks, and any other measurements which were considered of value in working up results have been carefully taken. The measurements taken are usually distributed over and under the surface of the floor tested in order to gain an idea of the changes occurring in different parts of the structure.

The above statement gives in general terms the features of any one of the tests dealt with in this paper. There are many difficulties to overcome and many chances for error. What follows is concerned mainly with the method of overcoming these difficulties and avoiding these errors. Most of the statements made represent the results of experience on previous building

tests. Some merely give ideas which it is believed if put into operation would be advantageous.

THE PLANNING OF A TEST.

Each test made will involve individual consideration of the choice of area to be loaded, the number and location of gauge lines and deflection points, the number of laborers required, the loading material to be used and its distribution, and provisions for storage of the loading material near the test area without appreciably affecting the stresses which are to be measured. Other matters will come up for consideration, but in the main they will not require different solutions for each test.

Choice of Test Area.—The area to be loaded should be chosen so as to fulfil the following conditions as completely as possible:

(*a*) It should be so located as to give conditions in the beams, slabs, columns, etc., as severe as will be found anywhere in the building when in use.

(*b*) It should be free from irregularities of construction.

(*c*) It should be as free as possible from disturbances of workmen.

(*d*) It should be as easily accessible to the loading material as possible.

In most cases some limitation is found on part or all of the conditions named. For example, in the test of the Wenalden Building it was impossible to find an area entirely free from irregularities of construction. An industrial track crossed one of the panels chosen, and the floor was thicker immediately under this track than at other places. On the edge of one or two of the panels tested, beams about an inch deeper than the regular beams were located. However, none of the measurements assumed to give typical results were taken in these panels, and it is believed that the stresses in the other panels were not affected appreciably by these irregularities. Again, in the test of the Franks Building it was not possible to choose a lower floor convenient to the loading material. An upper floor was used in order, during the course of construction, to make preparation for the test, thus avoiding digging in the concrete. However, this choice of a floor fulfilled one of the conditions mentioned, in that it gave a much more severe test of the columns than a test on a lower floor would have done.

Also, in the test of the Carleton Building at St. Louis the area to be tested was specified by the city building department, and there was no choice as to location on the part of those making the test.

Number of Measurements.—The number of measurements to be taken will depend upon the nature of the test, the number of observers, and the number of laborers. If the test is a part of a series by which it is expected to gain scientific information which will afford a basis for design, it is likely that it will be deliberate enough that a large number of measurements may be taken. Such tests were those of the Wenalden Building, the Franks Building, the Turner-Carter Building, and the Barr test panel.

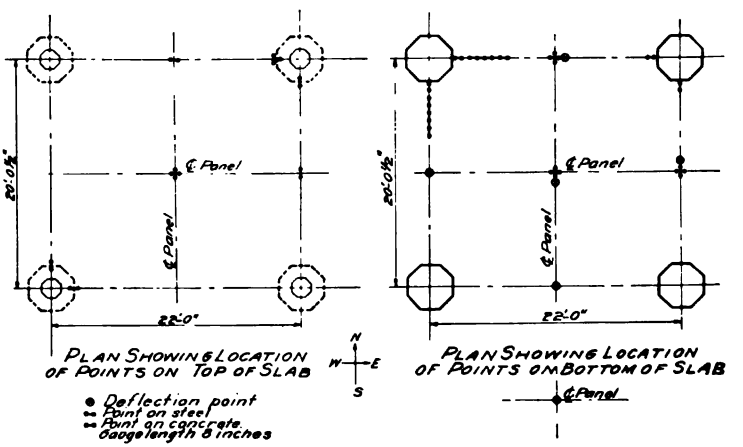

FIG. 2.—LOCATION OF GAUGE POINTS, CARLETON BUILDING.

If, on the other hand, the test has more of a commercial nature or is a utilization of the opportunity offered by the acceptance test to take some measurements which will show actual stresses, or if for any other reason the test is hurried, the number of measurements will necessarily be rather small. Of this class, the tests of the Carleton Building in St. Louis and of the Ford Motor Building in Detroit, Michigan, are good examples. Notice was given the engineers only about one day in advance that a test would be made on the Carleton Building. Permission was obtained from the contractor to expose bars for measurement in various points and to erect the necessary scaffolding. The meas-

urements were made more for the purpose of checking the analysis upon which the design was based than to form in itself a basis of design. Therefore comparatively few observation points were used. It is believed that this test is representative of the type of test which is practicable on a commercial basis, hence (by courtesy of the Corrugated Bar Company) a plan is given in Fig. 2 showing the points where measurements were taken.

Distribution of Measurements.—The arrangement of observation points will depend on what are the principal subjects for investigation in the test. Whatever the subject of study may be, the observation points should be arranged in such a way that a curve of deformations may be plotted against distance, showing

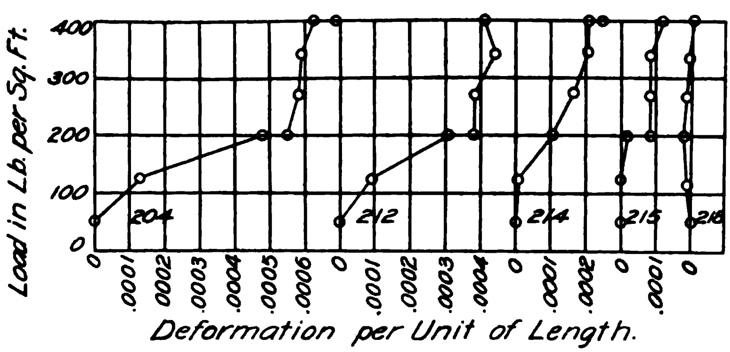

FIG. 3.—LOAD-DEFORMATION DIAGRAM FOR SERIES OF GAUGE LINES ON REINFORCEMENT, POWERS BUILDING.

a gradual progression from the condition at one part of the structure to the condition at another, for it is found that there are even under the most careful work, inconsistencies which will make the results look doubtful if standing by themselves. The points so arranged should be numerous near the place where the measurements of greatest importance are to be taken, so that the results will not depend upon measurements at a single point, or upon the average at portions of the structure supposed to be similarly situated but in different parts of the building where unknown conditions may actually cause a large variation in the phenomena of the test. It will not be possible to carry out this plan for all subjects of investigation, as the number of observations required

would usually be impracticably large. Such provisions may be made to cover the main lines of investigation, and isolated observation points may be used to gain information as to tendencies of other portions of the structure, but of course, less reliance must be placed on the results of the latter measurements than where the larger number of observations is made. It would be advantageous, as was done in the Powers Building test and also in the Barr test panel, for two observers to check measurements on the

FIG. 4.—LOCATION OF GAUGE LINES, POWERS BUILDING.

same points. One or both of these checks is very valuable in establishing the correctness of observations.

Figs. 3, 4, 5, and 6 illustrate the former method. Fig. 3 gives the load deformation diagrams for several gauge lines in the test of the Powers Building. Fig. 4 shows the location of these points with reference to the wall and a column. Fig. 5 shows the same data plotted as deformation against distance from the column instead of against load. It may be seen that the correctness of the load deformation curve for one of these points, if standing by

itself, might be doubted because of the complete change in the character of the curve at a load of 200 lb. per sq. ft. But when these deformations are plotted against distance, the results look so consistent that it is scarcely conceivable that they are seriously incorrect. In the test of the Wenalden Building very high deformations were observed in the concrete of the beams near the supports; so high that the results were doubted, and as the points on the load deformation curves were few and scattering, there was often room for doubt. For this reason it was considered especially important that evidence which would confirm or disprove this high compression in the concrete be obtained in the

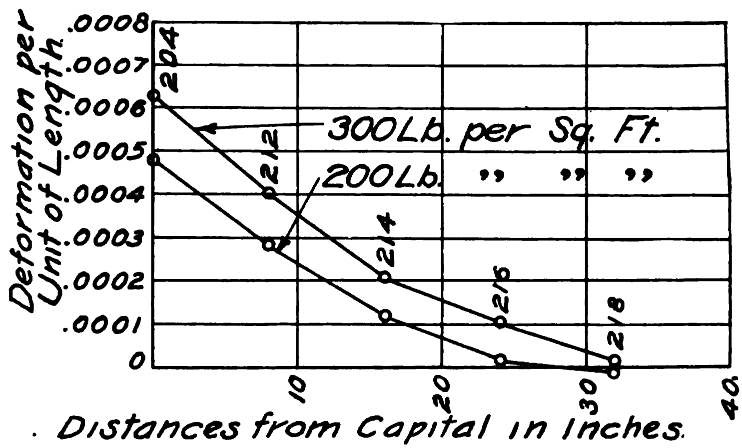

FIG. 5.—DATA OF FIG. 3 PLOTTED AS A DISTANCE-DEFORMATION DIAGRAM, POWERS BUILDING.

test of the Turner-Carter Building; accordingly the method of placing observation points at frequent and regular intervals along the ends of the beams was used. The deformations measured are plotted in Fig. 6 against the distance from the supporting column, and the results not only tend to show the correctness of these measurements but also to indicate that the high stresses observed in the beams of the Wenalden Building were actually present.

Subjects of Investigation.—In the tests discussed in this paper deformations have been measured with a view to obtaining information on each of the following subjects:

(*a*) The values of the moment coefficients at the center and support of the beam or slab under investigation.

(*b*) Relative moments at support for various conditions of fixity.

(*c*) The extent to which the floor slab acts as a compression flange of the floor beam to produce T-beam action.

(*d*) Bond stresses.

(*e*) Diagonal tension.

(*f*) Stresses in columns.

(*g*) Time effect under constant load.

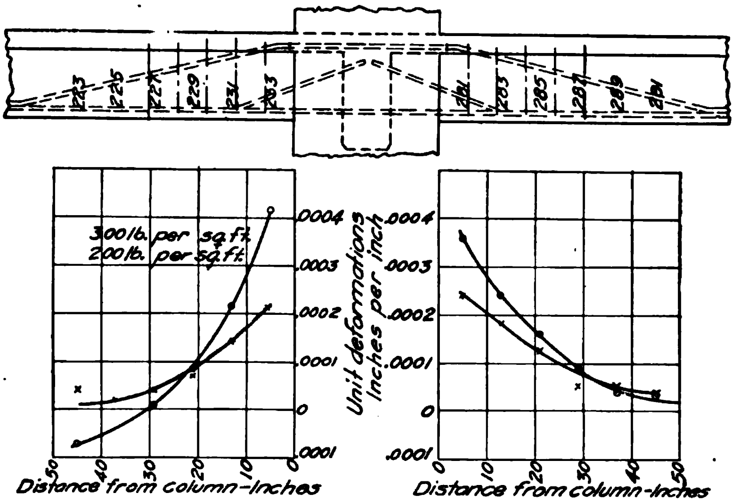

FIG. 6.—DISTRIBUTION OF COMPRESSIVE DEFORMATION IN BOTTOM OF COLUMN BEAM, TURNER-CARTER BUILDING.

(*h*) The lateral distribution of stress to parts of the structure entirely outside of the loaded area.

(*i*) The extent to which reinforcement stresses are modified by errors in the assumption that no tension is carried by concrete.

(*j*) Stresses in slabs of beam and girder construction.

Other subjects of investigation have received attention, but these are the most important ones. Some phenomena have been observed, offering additional problems, of which the determination of the amount of arch action present is probably the most important. It is important both in itself and because it is

intimately involved in the determination of moment coefficients. A further discussion of this problem is here given.

The attempts to determine moment coefficients have not been entirely successful due to errors of measurement and unexpected variations in similar parts of the structure remote from each other. The method has been to measure deformations on both reinforcement and concrete at the center and support, and from these measurements to determine the total resisting moment developed. Equating this resisting moment to a constant K times Wl a solution is made for the value of K. The indications that arch action has been present have so complicated this that even where measurements have appeared quite satisfactory, the uncertain amount of arch action entering has rendered the value of K uncertain.

A proposed method of determining the amount of arch action in any case is to make a special study of the deformations in a cross-section at the center of each beam across an entire panel. In this study, deformations should be observed on the reinforcement and at various elevations on the concrete so that the position of the neutral axis and of the center of gravity of tensile and compressive stresses respectively can be accurately located without dependence upon the law of conservation of plane sections. By this means it should be possible to determine if the sum of the compressive stresses is in excess of that of the tensile stresses. If so, the difference apparently must be the direct thrust due to arch action. The same study can be made, though not so satisfactorily, at the ends of the beams. This measurement of thrust will require observations on an extremely large number of gauge lines, and it would appear important to concentrate the greater part of the attention of the test on one panel.

If the floor be considered to be made up of strip-beams of differential width capable of transmitting shear from strip to strip, it is not necessary, for perfect beam action, that the sum of the tensile and compressive stresses on a cross-section of any one strip be zero. However, beam action does require that the sum of the tensile and compressive stresses on the total cross-section of the beam should be zero, and for this reason it is important to extend the investigation sufficiently to determine if appreciable deformations are continued out into the panel adjacent to the loaded area.

Laborers.—The number of laborers which can be used advantageously will depend on the distance from which the loading material is to be transferred, on the size and accessibility of the tested area, on the amount of work which can be done by them during the intervals between increments of loading while observations are being taken, and on the length of time required to take a series of observations. The handling of the labor should, if possible, not be left to the one in charge of the test, as proper

FIG. 7.—BRICK AND CEMENT AS A LOAD, WENALDEN BUILDING.

attention to the conduct of the test demands all of his time. In the tests included in this paper the number of laborers has varied between wide limits, from 5 or 6 in the Powers test to 30 or 35 in the Deere and Webber test.

Loading.—In the tests which have already been made, the following loading materials have been used: brick, cement in bags, loose sand in small boxes, sand in sacks and pig iron. The material used will almost always be that which is most easily available, because the moving of loading material from any

distance adds very greatly to the cost of the test. Leaving consideration of cost out of the question, sand in sacks seems to be the most satisfactory of the materials above mentioned for loading purposes. Some of the qualities of the materials mentioned are as follows:

(*a*) Brick: Brick spalls and chips in handling, covering the floor with dust and jagged particles which cause discomfort to the observer in kneeling to take observations. It is important to

FIG. 8.—SAND IN BOXES AS A LOAD, TURNER-CARTER BUILDING.

avoid this because discomfort necessarily decreases the accuracy of his observations. This might be avoided by sweeping, but in sweeping it is difficult to avoid getting dirt into holes where observations are to be taken, and this is just as troublesome as having the dirt on the floor. Fig. 7 shows the use of both brick and cement in the same test. Attention is called to the proximity of the cement sacks to the beams and girders of the floor above. In some cases the intensity of the load would be limited by the height of the ceiling if cement and brick are used.

(*b*) Cement: Cement sifts through the sacks and the sacks become untied, scattering cement on the floor, filling observation holes and causing much dust in sweeping or cleaning up. The dust is injurious to delicate instruments and annoying to observers and recorders.

(*c*) Loose Sand in Small Boxes: As sand is usually damp, it does not have the fault of causing dust and consequently is more easily cleaned up than the other materials mentioned. There are, however, other objections to it. In filling boxes it is difficult to avoid spilling the sand around and between the boxes, and con-

FIG. 9.—SAND IN SACKS AS A LOAD, BARR BUILDING TEST PANEL.

sequently filling the observation holes. On account of the great difficulty in removing loose sand without spilling a great deal of it, it is impracticable to take observations as the load is being removed, therefore it is necessary to remove in one increment the whole load from a given panel. Fig. 8 shows this method of loading.

(*d*) Sand in Sacks: Sand in sacks constitutes a very satisfactory loading material, Fig. 9. It was piled up to a height of about 9 or 10 ft. and very little inconvenience was caused by the sacks becoming untied or by spilling the sand. The worst difficulty encountered, and this exists with all materials handled in sacks, is that of the sliding of sacks on themselves when the load is piled

high. It can be seen in Fig. 9, above referred to, that bracing was necessary to prevent the sand from sliding together and filling up the aisles. It is a source of danger to those taking observations as, if a slide should occur, it would probably give very little warning and might catch the observer while in such a position that he could not escape. However, this objection would be likely to occur with any material which is piled as high as was that in this test. Under any circumstances it is necessary that care be taken and undue risks avoided.

(*e*) Pig Iron: Pig iron was used as loading material in the test of the Franks Building, Fig. 10. From the standpoint of

FIG. 10.—PIG IRON AS A LOAD FRANKS BUILDING.

the making of the test, the worst objection to it is that, as with the brick, small particles break off and cause annoyance to observers. This is less noticeable than with brick and in other ways pig iron is clean. It possesses the great advantage that with its use a very heavy load can be applied without piling the load extremely high.

Tin plate in boxes 2 ft. square, each weighing 200 lb., was to have been used in a building test. A more nearly ideal loading material would probably be hard to find, but unfortunately this test could not be carried out.

The intensity of the loading will depend mainly on the load

for which the building was designed. It will not be possible to make the load absolutely uniform, as aisles will be necessary for the purposes of (*a*) convenience in placing the load, (*b*) access to gauge lines for the taking of observations and (*c*) prevention of

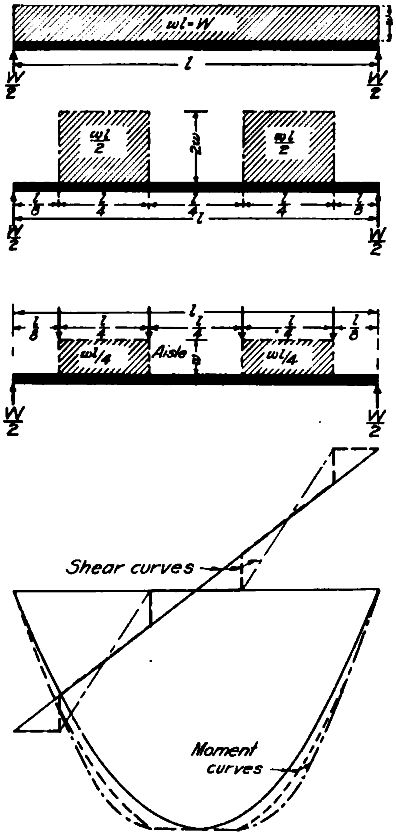

FIG. 11.—MOMENT AND SHEAR DIAGRAMS FOR THREE ARRANGEMENTS OF LOAD.

arching in the loading materials. It has been found that it is difficult to cover more than about 75 per cent of the actual area of the floor, and in many cases less than this will actually be covered. Hence in computing the probable height of the load, this fact must be taken into consideration.

Aisles should be so placed that the load, even though partly carried by arching of the material, will cause stresses in the floor which are approximately equal to and always as severe as those caused by an actual uniform load. Fig. 11 shows the moment and shear diagrams which would be obtained by loading a simple beam with a total load W distributed over the span in three different ways, as follows:

(*a*) Solid Line: Uniform load W, over full span.

(*b*) Broken Line: Same load W distributed over one-half of span, giving aisles of equal width at center and support.

(*c*) Heavy Dotted Line: Same load W distributed over one-half span, half of load being carried by arch action to ends of boxes (shown here as concentrated loads $W/8$), and the other half being uniformly distributed over the half span.

It will be possible in almost any test to arrange the loading material in such a way as to come within the limits outlined by the three arrangements of load assumed in Fig. 11, and it is seen that if this is done, the presence of the aisles or of arching to the sides of the boxes or piers, while not affecting the amount of the maximum moment and the maximum shear, would tend to cause them to exist over greater portions of the span than would be the case with an equal uniform load. In this figure aisles equal to one-quarter of the span have been assumed. In no case would they be as large as this, and, therefore, the moment and shear diagrams should actually conform even more nearly to those for uniform load than is shown in the figure.

Arrangement should be made, if possible, to store the loading material near the test area to hasten the work of applying the load after the test begins. The general rule has been to allow loading material to be stored as close as one full panel length from the test area, but the intensity of the storage load has been kept down as much as possible.

PREPARATION FOR THE TEST.

Cutting Holes in Concrete.—In all of these tests it is necessary to cut holes in the concrete in order to expose the reinforcement. Fig. 4 shows a hole cut in the concrete of the Powers Building where a series of measurements was taken on a rod passing through a column. This cutting has been best accomplished by the use

of a cold chisel with a very gradually tapering point. This is a task for common laborers and a long one for inexperienced men, but it has been found that a great deal of speed can be developed by practice, hence the importance of completing this part of the work with a single set of workmen.

A saving in mutilation of floors can often be effected by planning the test ahead of time and inserting plugs in the concrete during construction in the proper position for the gauge lines. Removal of the plugs after the concrete has set exposes the reinforcement without the use of a cold chisel. Likewise metal plugs may be set in the concrete at the proper positions for the measurement of concrete stresses and thus save cutting into the concrete to place compression plugs. The point has been raised that by preparation of this kind a chance is given to the contractor to know what panels are to be tested and thus to make the construction of that panel better than others. For this reason there is room for question as to the advisability of using this method. In most of the tests under consideration this point has been taken care of by the fact that it was not known until shortly before the test what area was to be loaded. It is believed that the saving thus effected is not generally sufficient to justify prejudicing the test by the use of this method.

Drilling of the gauge holes will be discussed under the subject "Instruments and Observations."

Scaffolding.—A platform supported on some kind of scaffold is necessary which will enable the observer to get close enough to the floor above to take observations of deflection and deformation. This should be at such a height that when the observer stands upon it the points where measurements of deformation are to be taken will be about one inch above his head. For flat slab construction this condition is easily obtained (see Fig. 12), but with beam and girder construction where there are measurements on beams, girders, and the floor slab, the heights of different gauge lines are so different that arrangement will need to be made for building certain parts of the platform higher than others (see Fig. 13). It is important that the elevation of the platform should be such that the observer can stand erect while taking the readings, and yet such that the instrument will not be too high for convenient and accurate observation.

Another framework for the purpose of supporting deflection apparatus under the points where measurements of deflection are to be taken is also necessary. In order that the movements of the observers upon the observation platform may not jar the deflection apparatus, the two frameworks must be built independently of each other. In all the tests which have been made, up to the present date, these deflection frames have stood on the floor and have been braced from one to the other in order to make a comparatively rigid framework. Fig. 12 shows scaffolding and deflection frames for the Franks test. An objection to this method of measuring deflections is that changes of humidity are likely

FIG. 12.—SCAFFOLDING AND DEFLECTION FRAMES, FRANKS BUILDING.

to change the length of the wooden posts used, and it is quite probable that an improvement could be made in the form of this frame. An arrangement which has been suggested consists of steel I-beams supported directly by the columns and carrying other steel framework on which can be placed the deflection apparatus. This would give more nearly a self-contained construction, and the changes of humidity and temperature would not change the deflection readings, except as the length of column between the platform thus built up and the floor above is changed.

Equipment.—The equipment will necessarily consist of the following: cutting and drilling tools, portable lights for throwing

light into observation holes, note books and facilities for doing drafting and for reducing data.

The cutting and drilling tools are sufficiently described in other paragraphs.

Some kind of a portable light is a necessity, as gauge lines are often located in dark corners and as observations may be taken at any hour of the day or night. The light shown in Fig. 20 is a

FIG. 13.—PHOTOGRAPH SHOWING VARIATION IN HEIGHT OF GAUGE LINES, TURNER-CARTER BUILDING.

hunter's acetylene light and is quite satisfactory. The light is attached to the forehead and may be thrown in various directions according to the setting of the clamp attachment. The acetylene tank may be attached to the belt or carried in the pocket.

Loose leaf note books should be provided in which the sheets are as large as can be conveniently handled and filed. The forms for record shown in Fig. 29 are very conveniently ruled in hectograph

ink and copied by means of a hectograph. Printed forms might be used, but so many differences in detail are made to correspond with the particular test in question that this would not be advisable as too few sheets of a single form would be required to justify the expense of having them printed.

For the most efficient work in computing results and making sketches for records, it is important that an adequate place be provided where some privacy may be had, where benches and drafting tables may be used and where instruments and other equipment may be kept. Fig. 14 shows the temporary office which was provided in the Turner-Carter Building test. This

FIG. 14.—TEMPORARY OFFICE, TURNER-CARTER BUILDING.

is one of the portable office shanties which the Company moves to places where work is being done. The photograph shows the interior of the office with the observers and recorders at work reducing the data of the test. This added equipment will add only slightly to the cost of the test and very greatly to the efficiency of the work. Special attention is called to it because there is a tendency to neglect this part and to think of it as only a secondary matter, whereas it should be considered as one of the most important pieces of equipment.

Summary of Test Data.—A summary of the main features of the building tests discussed in this paper is presented in Table I, as it is believed that the information given there will be of assist-

TABLE I—GENERAL DATA OF TESTS.

Building.	Type.	Test Area.		Loading Material.	Design Load, lb. per sq. ft.	Test Load, lb. per sq. ft.	Load Handled, tons.	Number of Gauge Lines.	Number of Observers.	Days Required for	
		In sq. ft.	Per cent of Total Area.							Preparation.	Test.
Deere and Webber	Flat Slab	2850	14.2	Brick and Cement	225	350	500	26	3	7	5
Wenalden	Beam and Girder	1800	3.5	Brick and Cement	250	400	420	52	3	8	7
Powers	Flat Slab	1070	40.0	Cement	200	400	174	66	2	–	—
Franks	Flat Slab	1959	16.6	Pig Iron	250	624	430	95	3	–	—
Turner-Carter	Beam and Girder	1690	14.1	Sand in Boxes	150	300	260	104	2	7	10
Barr	Reinforced Concrete and Tile	1270	100.0	Sand in Sacks	150	650	348	71	3	–	9
Carleton	Flat Slab	440	1.5	Brick	150	400	90	30	1	1	—
Larkin	Flat Slab	2450	10.0	Sand in Bins	225	618	500	268	3	5	6

ance in the efficient planning of and preparation for such a test. The following notes are in explanation of data given in this table:

The figures giving area of test show the total area of the floor covered and do not count any area twice even though loaded twice as was done in the Wenalden Building test. It does include the area of separate single panel tests which were made in the Wenalden and Franks tests.

The maximum test load in lb. per sq. ft. is given in the column under that caption. In some cases this was over only a part of the test area. The proportional parts of the test area having the maximum load applied were as follows: Wenalden 80 per cent, Powers 50 per cent, Franks 40 per cent, all others 100 per cent.

The column giving the amount of load handled includes the rehandling due to change of position of loads. The proportionate parts of the loads rehandled in this way were Wenalden 40 per cent, Powers 50 per cent, Franks 80 per cent. In all the other tests no load was rehandled.

The column giving the number of observers includes only those reading deformations. In the Wenalden and Powers tests another observer took deflection readings. In the Powers test and the Barr tests, almost all the deformation readings were taken by each of two observers, giving a larger number of gauge lines per observer than in the other tests.

III. INSTRUMENTS AND OBSERVATIONS.

Extensometers.—In the past the great obstacle to the measurement of deformations in building tests has been the difficulty of attaching the measuring instruments to either the reinforcement or the concrete on the flat surface of a floor, and recent tests show the necessity of making measurements of reinforcement deformation directly on the reinforcement. A satisfactory method of accomplishing this has been provided by the introduction of the extensometer invented by H. C. Berry of the University of Pennsylvania and adapted to this work by improvements made at the University of Illinois. This instrument is similar in some respects to the strain gauge designed and used as long ago as 1888 by James E. Howard, then Engineer of Tests at Watertown Arsenal.

The great value of this instrument in building tests lies in the following facts: (*a*) Its use makes it possible to take measurements

directly upon the surface of the reinforcement and concrete. (*b*) With its use there is no apparatus left in place to be damaged or disturbed during loading. (*c*) Due to the fact that it is portable, measurements may be taken in a large number of places with a single instrument. Measurements have been taken at as many

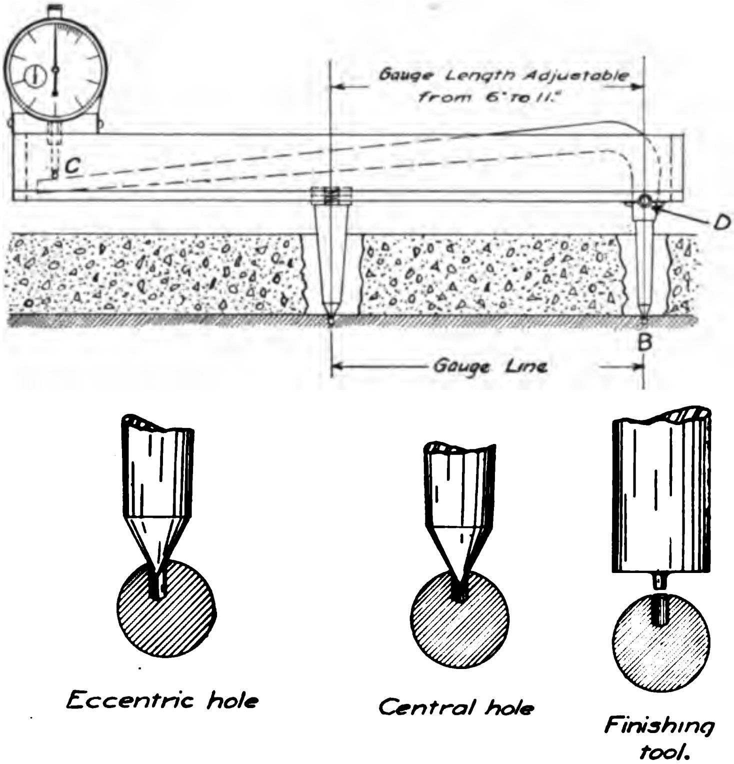

FIG. 15.—UNIVERSITY OF ILLINOIS EXTENSOMETER.

as 104 points in a single test. This would call for an outlay of from \$1200 to \$2500 for instruments if fixed instruments were used.

Fig. 15 shows the Illinois extensometer in its present form. Any movement of the point *B* due to a change in the length of the gauge line is transmitted to the Ames gauge through vertical movement of point *C*, by means of the leg *BD* and the arm *DC* pivoted at *D*. The Ames gauge is sensitive to a movement at *C* of .0001 in. The ratio of the length *CD* to the length *BD* is

approximately 5 and the Ames gauge is thus sensitive to a movement at B of .00002 in. (.0001 in. $\div$ 5). However, this must not be taken to mean that the extensometer possesses this degree of accuracy in measuring stresses, since some movement of the point of the leg at B is certain to result from variation in the handling of the instrument.

To obtain the exact ratio between movements at points B and C the instrument is calibrated by means of a Brown and Sharpe screw micrometer. For known movements of the point B readings of the Ames gauge are taken and a calibration curve plotted for the entire range of the instrument.

The first instrument of this type built by the Engineering

FIG. 16.—BERRY EXTENSOMETER.

Experiment Station of the University of Illinois was made by arrangement with Mr. Berry for the Deere and Webber test. It was designed by H. F. Moore and A. R. Lord, and was like the instrument in use at present except that it had a 15-in. gauge length and was made entirely of steel. Later on in making the instrument for general use aluminum was substituted for steel in order to reduce its weight and the gauge length was made variable from 6 to 11 in. Since then several minor changes have been made. The legs have been made stiffer in order to reduce the error due to unconsciously applied longitudinal thrust and the points have been made sharper in order to reduce the pressure required in seating the instrument. These improvements have reduced the probable error of observation considerably.

The extensometer loaned by Mr. Berry to the University of Illinois in 1910 and used in the Deere and Webber test is shown in Fig. 16. It differs from the Illinois instrument in that the movement of the multiplying arm is measured by means of a screw micrometer instead of the Ames gauge head, the point of contact of the micrometer plunger and the lever arm being determined by means of a telephone apparatus. The screw micrometer and the frame of the extensometer are insulated from each other and are connected with the poles of a small battery by means of copper wires. A contact between the plunger of the screw micrometer and the multiplying lever completes the circuit and the current set up produces a vibration of the diaphragm of the telephone apparatus

FIG. 17.—LATEST TYPE OF BERRY EXTENSOMETER.

carried on the head. This method of observation is very slow and the apparatus gets out of order very easily.

The use of the Ames gauge head instead of the screw micrometer and telephone apparatus adopted by Mr. Moore in the instrument used in the Deere and Webber test has greatly facilitated the use of the extensometer. The legs of this instrument also were made longer in order to adapt it to the measurement of deformations of reinforcement imbedded in concrete. Both of these modifications have later been used by Mr. Berry in instruments which he has put on the market.

The extensometer more recently designed by Mr. Berry is shown in Fig. 17. It is not different in principle from the one

just described. It differs from the Illinois instrument in the following details: (*a*) Instead of having a uniformly variable gauge length ranging from 6 to 11 in. it has two fixed gauge lengths of 2 in. and 8 in. respectively. (*b*) The instrument shown here has a multiplication ratio of two between leg and arm, and in order to make this ratio five (as in the Illinois type) it is necessary to use a leg which is only one inch long. With this arrangement the instrument can not usually be used for measuring deformations in reinforcing bars, owing to their depth of imbedment. (*c*) This instrument is put out with framework of Invar steel or aluminum. While Invar steel makes the weight

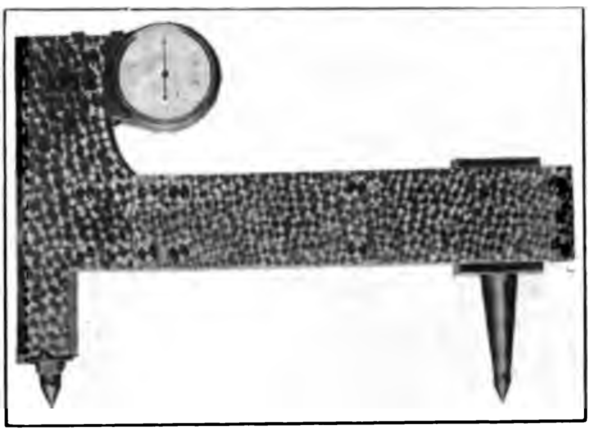

FIG. 18.—BERRY EXTENSOMETER AS MODIFIED BY TRELEASE.

somewhat greater than that of the aluminum instruments, it has the great advantage that so great dependence on an Invar steel standard bar is avoided and the study of the temperature changes in the reinforcement and concrete of the structure is accomplished with greater ease.

F. J. Trelease of the Corrugated Bar Company has designed an instrument of the Berry type and has used it in at least one test. This instrument, shown in Fig. 18, also has as its main feature a multiplying lever which actuates the plunger of an Ames gauge head. The principal difference between this instrument and the one shown in Fig. 15 is that the multiplying lever is

vertical instead of horizontal. Results have been obtained with it which do not differ much as to accuracy with those of the Illinois type of instrument.

Use of Berry Extensometer.—In obtaining good results with this extensometer, a great deal depends upon careful manipulation of it. Two things which are of great importance in this respect are (*a*) the preparation of the gauge holes, and (*b*) care and experience in the use of the instrument.

The exact gauge length is best secured by the use of some

FIG. 19.—INSTRUMENTS AND TOOLS.

kind of gauge marker such, for instance, as is shown in Fig. 19 used for marking points where gauge holes are to be drilled. In the work of the Illinois Engineering Experiment Station the holes are drilled with a No. 54 drill (.055 in. in diameter). At the beginning of the use of the Berry extensometer a number E countersink drill (approximately 3/32 in. in diameter) was used, but a smaller one seems to be better, because it is easier to get the properly finished hole, and because a slight eccentricity of the gauge holes on the reinforcing rod causes less error in manipula-

tion of the extensometer when a small drill is used. In the case of measurements on small rods also, the 3/32 in. drill cuts away a large percentage of the metal in the rods. Up to the present time, for drilling these gauge holes a breast drill has been used which is geared so that one revolution of the crank gives about $4\frac{1}{4}$ revolutions of the drill. In the hands of a skilled workman very satisfactory work can be done in this way, but where, as quite frequently will be the case, the drilling has to be done by persons not familiar with this kind of work, something better is needed. A drill driven by a flexible cable attached to a small electric motor giving a speed of rotation of 400 r. p. m. and upwards probably would be much better. Where high carbon steel has been encountered many drills have been broken and even when a hole was drilled a poor job has often been the result. After drilling the holes, the edges should be finished to remove the burr and to round off the sharp corners. The tool shown in Fig. 15 is designed to accomplish this purpose. Such a tool should not be a cutting tool but rather a wearing or polishing tool. A pointed magnet to remove steel dust and small fragments of steel torn off in drilling would be of use. It is hard to place too much emphasis on the proper preparation of gauge holes.

Standard Bar.—While the careful preparation of gauge holes is important, not less so is the use of a standard bar. The necessity for it was first found in the test of the Deere and Webber Building. Variation in temperature was sufficient to cause a change in the length of the instrument as great in many cases as that in the reinforcement due to the applied load. Hence it was found necessary to make observations on an unstressed standard bar showing any temperature changes in the length of the instrument. In this test a bar of about $\frac{5}{8}$-in. steel was used as a standard. It was protected from rapid temperature changes by imbedment in plaster of Paris, but kept on the floor where the test was being made. In this way it was expected to make the change in the length of the standard bar due to temperature variations about equal to the change in length of the reinforcement due to the same cause. To some extent this purpose was accomplished, but as the plaster covering was thin and not very dry the change in the standard bar must have been much more

rapid than that in the reinforcement. In the test of the Wenalden Building precautions were taken to imbed the standard bar in concrete. This practice has been kept up in tests made since then, and in addition standard gauge lines have been established in parts of the floor not affected by the load. These latter have been placed both on the reinforcement and in the concrete. Fig. 20 shows the taking of an observation on a standard gauge line in the Turner-Carter test. It can be seen that it is located in a

FIG. 20.—TAKING AN OBSERVATION ON STANDARD GAUGE LINE, TURNER-CARTER BUILDING.

part of the floor entirely away from the loaded area. The greatest development in the use of the standard bar has been in the frequency of reference to it and in the development of an exact system for the calculation of temperature corrections. It was previously noted that a steel instrument was used in the Deere and Webber test but that in the subsequent tests an aluminum instrument was used. Since the coefficient of expansion for aluminum is almost twice that for steel, it is apparent that dependence on the standard gauge line must have been of still

greater importance in the later tests. Difficulty was found in interpreting the notes taken on the Wenalden test, but the greater dependence on the standard gauge line and the more systematic use of it observed since then has very largely overcome this difficulty. Subsequent to the completion of the last building test participated in by the writer, standard bars of Invar steel have been secured. Invar steel has a coefficient of expansion only about one-sixth that of ordinary steel and its use as a standard bar makes it possible to eliminate from the results almost all the effects of temperature variation. If it is desired to determine how great are the temperature effects, a standard gauge line can be placed in the floor as before in such a position as not to be affected by the floor load.

It has been the practice in the more recent building tests for each observer to make observations regularly on two standard gauge lines. This is done so that one may form a check on the other. If only one were used, a large accidental change in the readings due for instance to sand in the gauge holes might be mistaken for a temperature effect. If two standards are used, any such accidental change as the above would seldom be the same in both, and the error would be detected. An accident to the instrument would probably cause the same change on both standard gauge lines and the use of the two standards would not help to detect this kind of an error. However, such errors are usually so large as to be apparent in any standard reading and are infrequent as compared with errors due to filling of the gauge holes.

Deflection Instruments.—In the building tests described in this paper deflection instruments of two types have been used, one being that used by the Illinois Engineering Experiment Station and the other that used by the Corrugated Bar Company. The former, shown in Fig. 21, consists of a screw micrometer head of 1 in. travel, connected in tandem with an Ames gauge head micrometer of $\frac{1}{2}$ in. travel. The screw micrometer is designed to cover large variations in deflections, and the Ames gauge head, small ones. Fig. 21 shows also the method of using this deflectometer. A plate, having a $\frac{1}{2}$-in. steel ball attached, is plastered to the surface, deflections of which are to be measured. A $\frac{5}{8}$-in. bolt, which has a steel ball inserted into its upper end,

is set into a wooden block (part of the deflection framework) in such a way that its elevation can be adjusted to give any desired zero reading of the deflectometer. Thus at the beginning of a test all the zero deflection readings can be determined so that for a considerable length of time all the change in deflection will be shown on the Ames gauge without any change of the screw micrometer. As larger changes take place, a second setting of

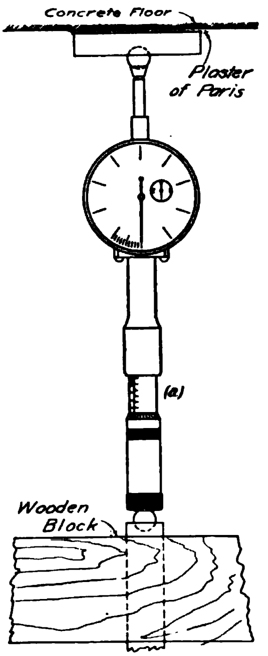

FIG. 21.—DEFLECTOMETER, UNIVERSITY OF ILLINOIS.

the screw micrometer will probably be necessary. The great advantage of this instrument is the rapidity with which it can be used. It has been found to work very satisfactorily in most respects. A shortcoming, however, has been the lack of a revolution counter on the Ames gauge so that in case of large changes of deflection it is possible to make an error of as much as 0.1 in. in interpreting the readings, though this is very unlikely. This instrument was last used in the Turner-Carter test and since then

an Ames gauge head, which has a revolution counter, has been provided for it, so that the difficulty here mentioned is not likely to occur in the future.

The deflectometer used by the Corrugated Bar Company is shown in Fig. 22 and consists of a screw micrometer depth gauge by means of which distances for varying loads are measured between the stationary frame and a point on the beam or floor

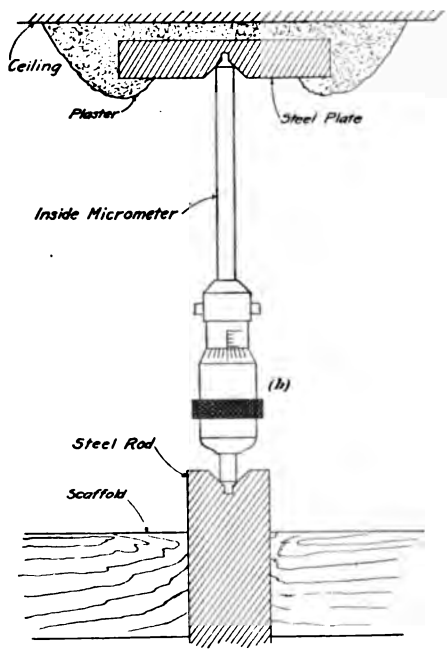

FIG. 22.—DEFLECTOMETER, CORRUGATED BAR COMPANY.

slab. It has the advantage over the one previously described that actual distances are measured instead of changes in distance, so that if the complete reading is taken each time, there is no possible way of misinterpreting results. It has also the advantage of a much larger range of measurement. In the Barr panel test a gross deflection of more than 3 in. took place. As the Illinois type of deflectometer has a range of only 1½ in. it could not have been used in this test. This, however, is more than

would often if ever occur in the test of a building. Its disadvantage is that it requires a longer time to make an observation than does the deflectometer previously described.

Observers.—Observers should be experienced in the use of the Berry extensometer before undertaking work on a field test. The chances of error in the manipulation of the instrument are large and as a rule the deformations measured are small, so that the error is likely to be quite a large proportion of the total measurement; hence it is important to reduce errors to the lowest possible limit.

Extensometer Observations.—If the observations at zero are equally as good as other observations, a curve may be drawn through all the points of any load-deformation diagram after the test is completed, weighting the zero observations equally with the others and the zero point shown by the most probable curve should be used as the origin. This method involves waiting until the completion of the test to draw these curves. It would be better to spend much more time on the zero observations, in order to make them reliable, than is paid to any other series. By this means a check can be had upon the action of the structure as the test progresses and the construction of the most probable curve will be made more simple. To do this it is essential that several complete series of zero observations should be taken with no load on the floor, and it would be well to repeat this through considerable range of temperature to study temperature effect on the reinforcement and on the concrete. This study was attempted in the Deere and Webber test, but the changes both in instruments and in reinforcement were included in the measurements and could not be separated, so no definite conclusions could be drawn. However, with an Invar steel standard bar or with an instrument made of Invar steel these two kinds of changes can be separated and to some extent at least the effect of temperature determined.

In taking an ordinary observation about five readings should be averaged. In all of the building tests which have been made, individual extensometer readings were recorded, but in laboratory tests the practice of averaging the results mentally has been adopted. This gives very satisfactory results for laboratory tests and saves a great deal of time. It is possible that this practice

could be adopted for field tests also. It would save time on a test and with a good recorder the calculations could be kept up with the observations. In the more recent building tests the practice followed in obtaining readings for any observation has been to reject all readings until 5 consecutive ones have been obtained which agree within .0004 in. These five consecutive readings then are averaged to form an observation.

Deflectometer observations have been sufficiently discussed in the description of the deflectometer and will not be taken up again here.

Observation of Cracks.—Up to very recently the observation of cracks has been considered one of the most important features of a test, and if carefully done it may yet add considerable to the confidence in the results. These observations should be made and recorded for zero load and at each increment of load. This is one of the most tedious parts of the test, and to carry it out faithfully requires a great deal of patience. The examination should be minute and very thorough. One who is not familiar with this kind of work will be likely to miss important indications and careful supervision should be maintained over this part of the investigation.

Special attention has been called to observation of cracks because of incorrect ideas which apparently prevail with regard to them. It seems to be the idea of some engineers that the type of construction advocated by themselves is immune from cracks. When it is remembered that plain concrete fails in tension at a unit deformation of about .0001, it is apparent that cracks must form when the stress in the reinforcement is such as to correspond with this deformation, or at about 3000 lb. per sq. in. At this stage the cracks are often too small for detection with the naked eye, but almost always very fine cracks are found at stresses ranging between 3000 and 10,000 lb. per sq. in. Thus to report for a floor loaded to twice the designed load that no cracks were observed is to admit one of three things, namely, that an excess of reinforcement was used, sufficient care in taking observations was lacking, or that not all the facts of the case were reported.

It should be borne in mind that the cracks referred to in this paper are often extremely minute and usually are not visible to a casual observer. Frequently cracks have been traced with

a lead pencil to make them distinct for the purpose of sketching, and it seems apparent that some persons visiting the test have mistaken these pencil marks for large cracks. At any rate reports have been circulated as to the existence of large cracks in a test where to the writer's personal knowledge there were none.

ACCURACY OF DEFORMATION MEASUREMENTS.

Probable Error.—The ratio of multiplication in the Berry extensometer is not exactly equal to the ratio of the length of the arm to the length of the leg, the error being due to the fact that the plunger of the Ames gauge head does not always travel in a line perpendicular to the multiplying lever. However, calculations show that this approximation results in an error in the measurement of reinforcement stresses equal to only about one-quarter of one per cent for an extreme case. It may be seen later that errors of observation are large enough in proportion that this error can be neglected.

In forming a basis for a conclusion as to the accuracy of the figures given out as results of tests, use has been made of the check readings taken by two observers on the same gauge lines and of calculated probable error of the means of five readings. While it is possible to calculate with some accuracy the probable error of replacing the instrument on the same gauge line time after time at one sitting, it is very difficult to determine the error caused by gradually cramping the quarters of the observer as the loading material piles up. A determination of errors based on independent checking by a second observer should be expected to eliminate to a large extent errors of all kinds and the greatest dependence should be placed on this kind of results.

In the test of the Powers Building most of the observations taken were checked by a second observer and some of the results are shown in the load stress curves of Fig. 23. The values shown in solid circles were observed by F. J. Trelease and those in open circles, by the writer. The zero reading for the latter is in each case at a load of 50 lb. per sq. ft., and in order to make a direct comparison of results, all these curves must be set over so that their zeros coincide with the stress values at 50 lb. per sq. ft. of Mr. Trelease's curves. Having made this correction the average variation between all the comparable points is about 670 lb. per

sq. in. (.0000223 unit deformation), which amounts to a probable error of approximately ±340 lb. per sq. in. (±.000011 unit deformation).

Fig. 24 shows the results of a series of measurements taken in the same way on the upper and lower surfaces of a 4 x 4 in. timber beam loaded with sacks of sand on a 12-ft. span. The points in open circles represent measurements on the top surface and those in crosses on the bottom surface. Determined in the same way,

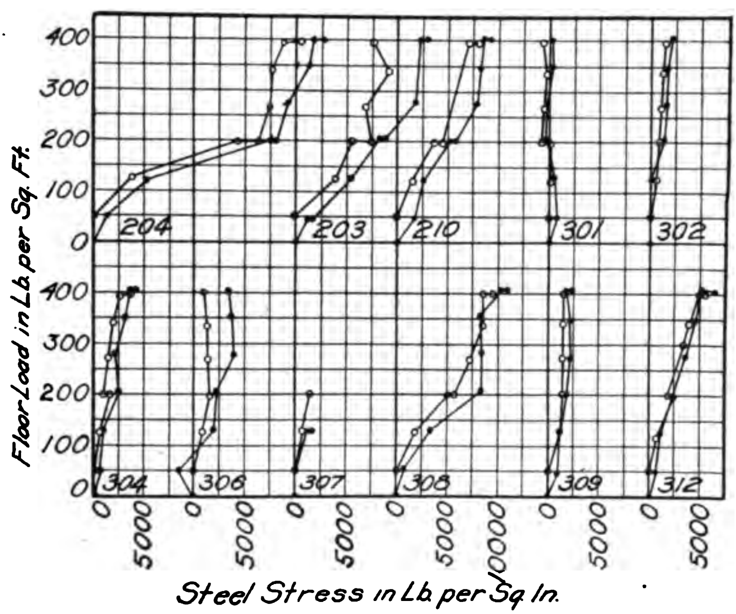

FIG. 23.—LOAD-STRESS DIAGRAMS OF TWO OBSERVERS, POWERS BUILDING.

these measurements show an average probable error of approximately ±.000017 unit deformation. As previously stated, these check measurements must be taken to give results more applicable than calculations of probable error of the mean of a group of readings. However, it may be expected that where an increase in accuracy of setting the instrument is found, a decrease in error due to cramped quarters, etc., will also be found. In Fig. 25 is given a curve which shows for each of four building tests the probable error of the average of five readings. Each plotted point

is the average of the probable errors calculated for six different gauge lines at a given load. What this diagram may be expected to show is the improvement in results with increased experience rather than the actual value of the probable error. The marked improvement in results shown here is due in part to increased skill in the observer and in part to improvement in the instrument itself. Fig. 26 gives a curve showing deformations in a bottom bar of the Barr test panel as shown in the sketch. The points shown

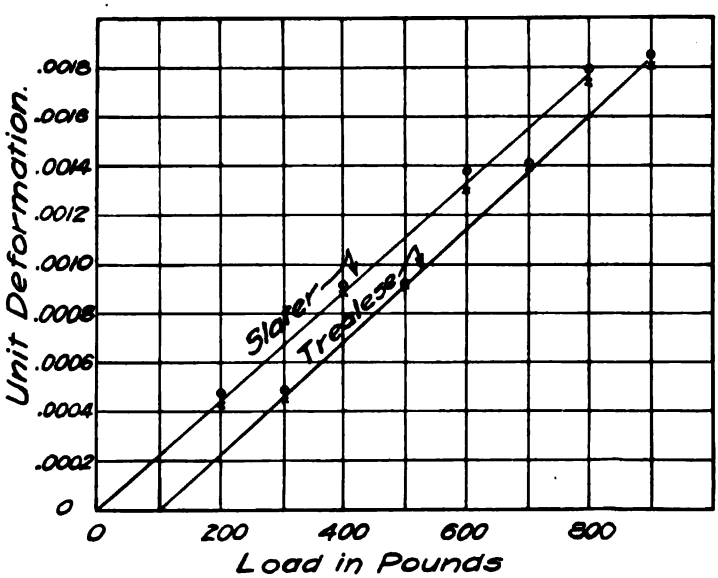

FIG. 24.—LOAD-DEFORMATION DIAGRAMS OF TWO OBSERVERS; TEST OF A 4 X 4-IN. TIMBER BEAM.

as open circles are for a load of 590 lb. per sq. ft. and solid circles are for a load of 615 lb. per sq. ft. This is the best curve the writer has been able to obtain on any building test and it can not be taken as representative, but rather to illustrate what may be obtained under the best conditions. The regularly varying differences for a small difference of loads indicate that the stresses must have been determined correctly within a very small range.

A study of probable error was made in the Turner-Carter test by the use of a series of 100 observations taken by each of the

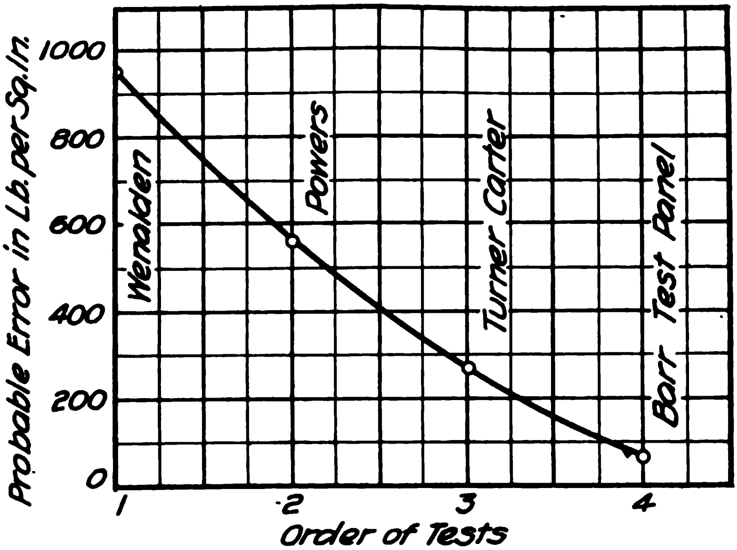

FIG. 25.—AVERAGE PROBABLE ERROR FROM FOUR BUILDING TESTS.

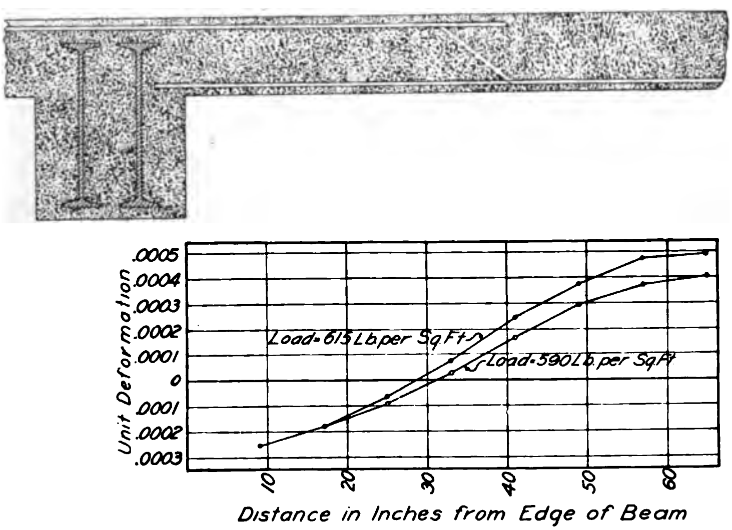

FIG. 26.—DEFORMATION ALONG BOTTOM REINFORCING BAR, BAR PANEL TEST.

two observers on two gauge lines selected as likely to give the most and the least accurate results. The results of this study are given in Table II.

TABLE II—PROBABLE ERROR OF THE AVERAGE OF FIVE CONSECUTIVE READINGS.

	Observer	Gauge Line. 1	Gauge Line. 2	Gauge Line. Average.
Unit deformation..................	H. F. Moore	.00000687	.0000106	.00000873
	W. A. Slater	.0000043	.000014	.0000091
Stress in reinforcement in lb. per sq. in.	H. F. Moore	206	318	262
	W. A. Slater	130	435	282

While these measurements were not all on reinforcement, the probable error has been reduced to terms of stress in reinforcement for convenience of interpretation. It is very interesting to note that the average probable error of ±282 lb. per sq. in. agrees very well with that for the Turner-Carter test as shown in Fig. 25. The same observer took the data in both cases, but the data for the value shown in Fig. 25 are taken directly from the records of the test and represent the condition on six typical gauge lines. The method of obtaining the values given in Table II is explained above.

From the data in hand it seems safe to conclude that for ordinary conditions stresses in reinforcement can be measured to the nearest 1000 lb. per sq. in., though in the past there have been some glaring failures to obtain as great a degree of accuracy as this. The advantage of further increase in accuracy of results lies in the determination of the relation of parts of the structure.

Effect of Changes in Temperature on Accuracy of Results.—Changes of temperature will give measureable changes of length in reinforcement, in concrete and in instruments made of ordinary materials. In most of the building tests corrections have been made for the changes in the instrument due to changes in temperature by means of observations on standard unstressed gauge lines chosen to represent as nearly as possible the conditions of the reinforcement and the concrete in the part of the structure tested. The method of calculating this correction will be described

below. It is there mentioned that in distributing the corrections found by reference to the standard bar, a linear variation from the time of one standard observation to the time of the next standard observation was assumed. Some observations have been made to determine the correctness of this assumption.

To determine the amount of change in length of an aluminum extensometer covered and uncovered, a test was made in which the two instruments were suddenly exposed to a change of temperature of 60 deg. F. A covering which consisted of a double layer of rather heavy felt protected one of the instruments from too sud-

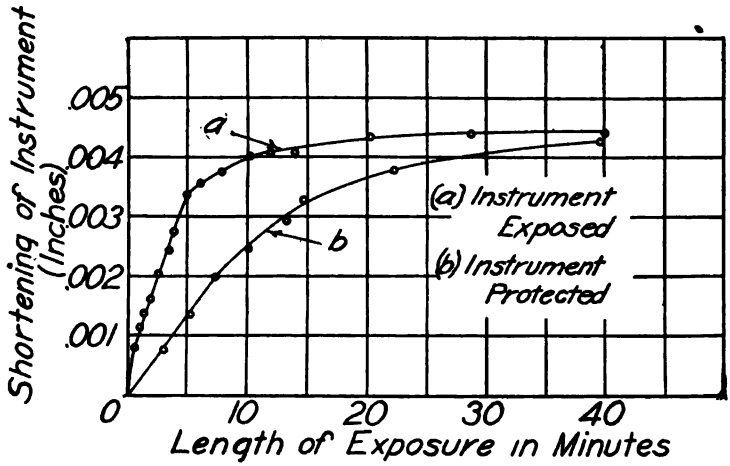

FIG. 27.—DIAGRAM SHOWING CHANGE IN LENGTH OF INSTRUMENTS DUE TO CHANGE IN TEMPERATURE.

den a change in temperature. The other instrument was entirely unprotected. The results of this test are shown in Fig. 27 with the change of length of the instrument plotted as ordinates against time as abscissas. For these measurements a standard bar of Invar steel was used. The coefficient of expansion of this being very small, the change of length measured must have been almost entirely that in the instrument. The curve shows that for an instrument not insulated from temperature changes only about five minutes is required for the instrument to come to the temperature of the air. For the insulated instrument about 20 minutes was required. This may be interpreted to mean that if an unpro-

tected instrument is used, readings on the standard bar should not be more than five minutes apart. With an instrument protected as was this one, intervals of 20 minutes would not be too much. The amount of change for the case shown here is extreme as the instrument was suddenly exposed to a change in temperature of about 60 deg. F. This range would seldom be found, and the length of time required to make the change for a smaller difference of temperature may be less but probably would not vary much for other ranges of temperature. It may be concluded

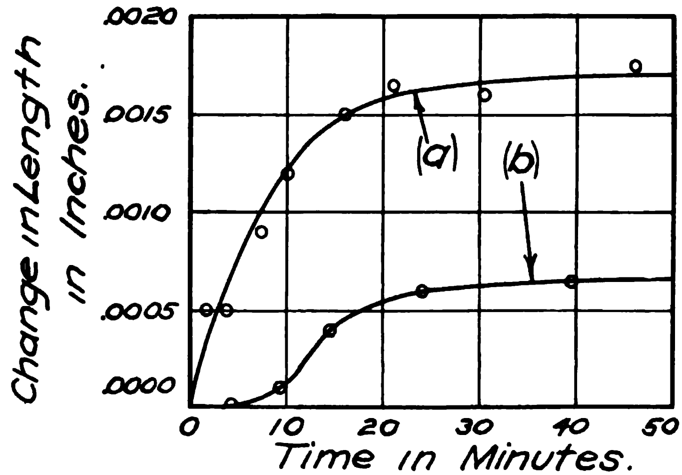

FIG. 28.—DIAGRAM SHOWING CHANGE IN LENGTH OF A STEEL BAR DUE TO CHANGE IN TEMPERATURE.

(a) $\frac{3}{8}$-in. square bar exposed.
(b) $\frac{3}{8}$-in. round bar embedded in concrete.

that the method used for distributing the correction is justifiable, since the instrument was protected from sudden change of temperature and the observations on standard bars were usually at intervals not greater than 20 minutes.

Temperature Effect on Reinforcement.—The above test shows the effect of change in temperature on the instrument. Another test was made to determine the effect of change in temperature on reinforcement imbedded in concrete and also exposed to the air. A $\frac{3}{8}$-in. square bar of steel entirely unprotected from temperature

changes and a $\frac{3}{8}$-in. round bar imbedded in 1 in. of concrete were exposed to a sudden change of temperature of about 43 deg. F. Measurements were taken on a 6-in. gauge length of each bar at very short intervals of time. The results are shown in Fig. 28. The results of this single test must be used with caution as the total measurements were very small and a small error would show up very plainly. However, the curve for the imbedded bar agrees in its general characteristics with some of the results obtained by Professor Woolson on "Effect of Heat on Concrete" reported in 1907.* The test indicates that for this range of temperature rather rapid changes may be found in the reinforcement, corresponding with stresses of about 9000 lb. per sq. in. and 3000 lb. per sq. in. respectively for exposed reinforcement and that protected as in this case. The range of temperature is extreme and the size of bars smaller than is often found in floor construction, therefore the results found in tests would probably be less extreme. However, this indicates the necessity of attempting to eliminate from the results of the test the effect of temperature changes, especially if the stresses measured are small.

IV. RECORDS AND CALCULATIONS.

Since the beginning of the use of the Berry extensometer for testing purposes, as much development has been made in the keeping of notes as in the use of the instrument. Because of a lack of completeness of notes the advantages of the use of the standard bar were not fully realized for some time. Only after the method of keeping notes had been highly systematized was it possible to properly make the corrections which observations on the standard bars indicated should be made. During the time of placing an increment of load the recorder will have considerable time in which to be working up results of the series of observations taken at the previous increment of load, and as the method of making these calculations is quite intricate, a man is required for this work who has ability to do more than merely record. It is important that calculations should be kept up as the work progresses, because it can be done with less labor then than at any other time and because it will be of value to know as the test progresses what results are being secured.

* See *Proceedings*, Vol. VII, *Am. Soc. for Test. Mats.*

RECORDS.

It is very important on account of the great number of observations taken (about 12,000 in the Turner-Carter test) that all records be arranged systematically. The following points are mentioned as being important in this connection. (*a*) In the field test individual readings should be recorded and their average used as a single observation. The proposed abridgment of this procedure (see p. 204) should be considered as a suggestion for later development. (*b*) Recording readings in the order of their size will assist the recorder in obtaining the correct readings and in rapidly obtaining the average. (*c*) The exact sequence of observations should be maintained in the records as the calculation of corrections depends largely on this.

Fig. 29 shows a form for the recording of original readings and the results calculated from them.

CALCULATIONS.

The calculations of corrections and applying them to the results makes the reduction of data rather intricate. This work has been reduced to a definite form shown in Fig. 30. In this form the zero length of instrument (see definition, p. 172) is assumed as correct and is used as a standard of reference. The corrections are distributed among the gauge lines as though the change in the length were a linear function of the time from one standard bar observation to the next one. These assumptions do not entirely accord with the facts but have been satisfactory as a working basis. Any other standard bar observation than the zero length would do as well for a standard of reference except for matters of convenience.

V. COST OF THE TESTS.

An attempt was made to get information by which the cost of the tests could be estimated, but it is found that from the data on hand no finely drawn conclusions are warranted. The costs of the tests enumerated here range from about \$50 to as much as \$2000, depending on the nature of the test and the expenses for railroad fare, hotel bills and pay for expert assistance. In the case of \$50 the cost is only that in excess of what would have

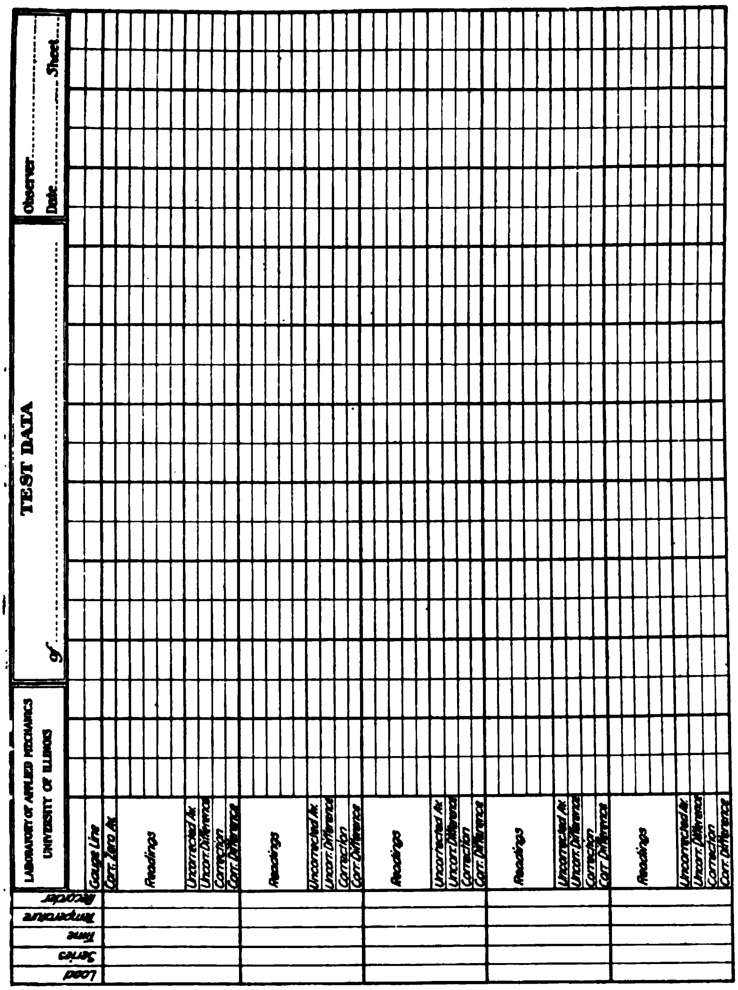

FIG. 29.—FORM FOR RECORDING ORIGINAL READINGS AND CALCULATED RESULTS.

Load	Interval	0		1	2	n-2	n-1	n	
	Gage Line	Standards a	Standards b	Arbitrary Numbers for Gage Lines				Standards a	Standards b
Dead Load only	Uncor. Average	S_a	S_b	R_1	R_2	R_{n-2}	R_{n-1}	S'_a	S'_b
	Correction	0	0	$\frac{1}{n}\frac{C'_a+C'_b}{2} = C_1$	$\frac{2}{n}\frac{C'_a+C'_b}{2} = C_2$	$\frac{n-2}{n}\frac{C'_a+C'_b}{2} = C_{n-1}$	$\frac{n-1}{n}\frac{C'_a+C'_b}{2} = C_{n-1}$	$S_a - S'_a = C_a$	$S_b - S'_b = C'_b$
	Corrected Zero. Av.			$R_1 + C_1 = A_1$	$R_2 + C_2 = A_2$	$R_{n-2} + C_{n-2} = A_{n-2}$	$R_{n-1} + C_{n-1} = A_{n-1}$		
Any Live Load P	Uncor. Average	s_a	s_b	r_1	r_2	r_{n-2}	r_{n-1}	s'_a	s'_b
	Uncor. Difference			$A_1 - r_1 = d_1$	$A_2 - r_2 = d_2$	$A_{n-2} - r_{n-2} = d_{n-2}$	$A_{n-1} - r_{n-1} = d_{n-1}$		
	Correction	$S_a - s_a = C_a$ *	$S_b - s_b = C_b$ *	$C_{ab} + \frac{1}{n}(C'_{ab} - C_{ab}) = C_1$	$C_{ab} + \frac{2}{n}(C'_{ab} - C_{ab}) = C_2$	$C_{ab} + \frac{n-2}{n}(C'_{ab} - C_{ab}) = C_{n-2}$	$C_{ab} + \frac{n-1}{n}(C'_{ab} - C_{ab}) = C_{n-1}$	$S - s'_a = C'_a$ †	$S_b - s'_b = C'_b$ †
	Corrected Difference			$d_1 - C_1 = e_1$	$d_2 - C_2 = e_2$	$d_{n-2} - C_{n-2} = e_{n-2}$	$d_{n-1} - C_{n-1} = e_{n-1}$		

* Let $\frac{C_a+C_b}{2} = C_{ab}$

† Let $\frac{C'_a+C'_b}{2} = C'_{ab}$

FIG. 30.—FORM SHOWING METHOD OF REDUCING DEFORMATION DATA.

been necessary for the acceptance test of a building. It does not include the salaries of the testing engineers and as there were no hotel expenses this cost is perhaps $150 less than what could be ordinarily expected for such a test. The data in hand indicate that a test as extensive as several in this series have been would be likely to cost from $1500 to $2000, but that one of the type represented by the Carleton test may be made at an expense slightly above that of the acceptance test so frequently required.

For a very slight additional cost, measurements of stresses in a building floor may be made at points of especial interest during the progress of the load test which is often required as a condition of acceptance.

The stage has been reached in the investigation of reinforced concrete where building tests may be expected to contribute information of great value to the designer and builder in reinforced concrete. The main feature of such tests should be the measurement of stresses, but information as to the location and size of cracks will be of great value in checking the results if the examination for cracks is conducted with sufficient care and minuteness. There is need for increasing as much as possible the accuracy of deformation measurements and experience in the use of the instrument is gradually accomplishing this. All the confirmatory evidence possible on the correctness of results should be obtained.

THE DESIGN OF CONCRETE FLAT SLABS.*

By F. J. Trelease.†

The rapid introduction and development of flat slab floors of reinforced concrete is sufficient evidence of the great importance of this type of construction. In fact, the demands of owners and builders for flat slabs are so insistent, that in spite of the lack of reliable methods of design, many acres of such floors are built every year. The insistence of this demand has led many engineers to advance theories of the structural action of flat slabs, and these theories yield perhaps the most striking contrasts and disagreements to be found in modern engineering practice. The reason for this is largely because the flat slab is an extreme example of a redundant structure, and its mathematical analysis has so far been based upon certain arbitrary assumptions which vary in the different analyses. The differences in these assumptions are so great that the practical application of the theories founded upon them to flat slabs of reinforced concrete yield results varying by about four hundred per cent. This variation leads one to class the flat slab as beyond the range of pure analysis and one must look to experimental engineering for a satisfactory solution of the problem.

The usual load tests of completed structures may be dismissed at once as being entirely inadequate as a basis of design. In such tests usually but one panel of a structure is loaded, which does not give maximum stress conditions in a flat slab floor. Even if several panels be loaded and the test carried to destruction it will at best only roughly indicate the stresses at the weakest point of the structure under the scheme of loading employed and cannot give any information as to the economy of the design.

A very recent and more adequate form of test on completed structures is that in which several panels of a building are loaded, not necessarily to destruction, and in which the actual elastic deformations of both reinforcement and concrete are measured

* Advance Review of a Thesis presented to Washington University for the degree of Civil Engineer.

† Engineer, Corrugated Bar Company, Buffalo, N. Y.

by extensometers. The results obtained in such a test would form a satisfactory basis for the design of similar structures if one could eliminate from the results the effects of tension in the concrete, arch action, annular slab action and many other indeterminate factors, which enter largely into all tests of reinforced concrete. These effects are so indeterminate that they cannot be satisfactorily eliminated, thus rendering results of tests on completed structures more or less unsatisfactory as a basis of design. The average engineer hesitates to count on these factors in designing and even if he had no personal objections to their use, none of the building laws or regulations will permit of any allowance being made for them.

It would seem, then, that for satisfactory solution of the problem there must be obtained an empirical analysis of the structural action of flat plates derived from experiments on plates of homogeneous material.

Such an analysis could be then used in the design of flat slabs of reinforced concrete, using any desired combination of unit stresses and making such allowances for tension in the concrete, etc., that personal judgment or various building regulations might dictate.

It is the purpose of this paper to describe and give the results of such an empirical analysis based on experiments conducted by the author in the Research Department of the Corrugated Bar Company, under the general direction of Mr. A. E. Lindau, Chief Engineer. This work was started early in 1910, and has continued almost uninterruptedly to the present time.

A very interesting and exceedingly simple little experiment was made on a sheet of heavy cardboard fastened over twelve spools representing columns. By pressing the fingers on this little model at various points much was learned as to the general action of flat plates. For instance, one could press upon the center of several panels and note that the surface of the model was convex upwards at right angles to the lines joining the columns and forming the sides of the panels, showing that tension in the top face existed at that point, although none of the systems of reinforcement then in use provided resistance to these stresses.

The work most productive of results was a series of experiments on rubber models of flat slab floors. Rubber was chosen

for the models as best fulfilling the requirements of reasonable homogeneity, low modulus of elasticity, and the ease with which all its physical properties could be determined in the laboratory. The models used consisted essentially of a plate of pure gum rubber, fixed so as to form the top of an air-tight box containing the proper number of $1\frac{1}{4}$ in. round columns to divide it into nine panels, each 8 in. square.

The method adopted for applying load to the model was rather unique and extremely satisfactory, as it not only permitted the intensity of the load to be easily and accurately read, but

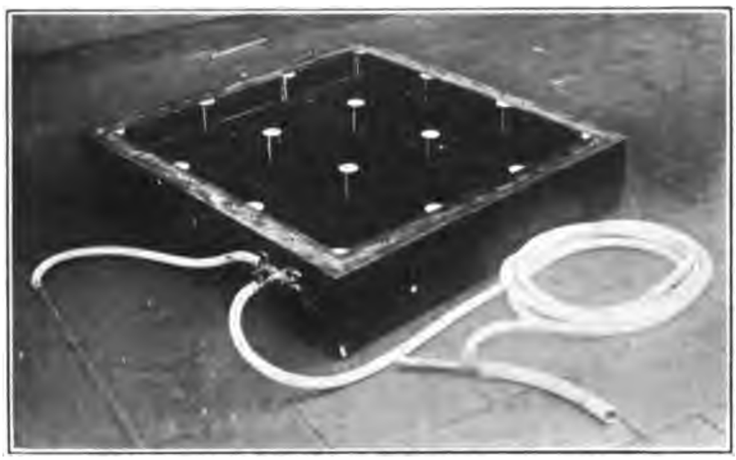

FIG. 1.—BOX AND ASPIRATOR FOR RUBBER MODEL OF FLAT SLAB FLOOR.

insured absolute uniformity of distribution, and at the same time left the entire upper surface of the slab unobstructed and free for observations and measurements. A partial vacuum was formed in the box, thus obtaining on the face of the plate the pressure of the atmosphere which was read by a simple U-tube manometer. The box and aspirator are shown in Fig. 1.

In the first series of experiments the rubber plate used was 0.34 in. thick. A stress strain diagram obtained from a strip cut from this plate is shown in Fig. 2. At first the scope of this series was limited to the measurement of the shape of the elastic surface of the plate under stress. With this end in view, deflection

readings were taken at numerous points in the various panels. In some cases these were taken at points only one-twentieth of the span apart.

The values of these readings were then averaged, grouping those which through symmetry should be alike, and the results plotted. Equi-deflection lines were then drawn, giving contour

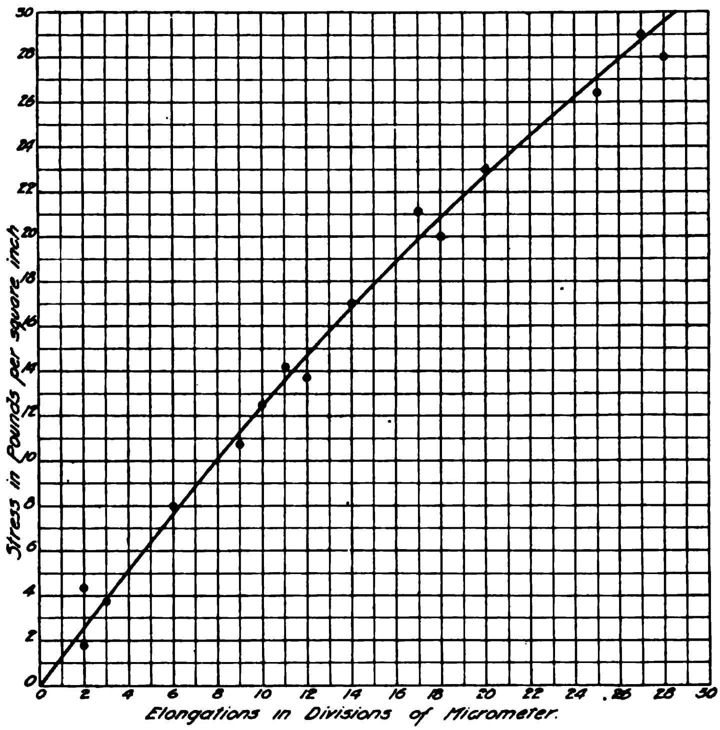

FIG. 2.—STRESS STRAIN DIAGRAM FOR RUBBER.

maps of the surface. Fig. 3 shows such a map for one-quarter of the model, while Fig. 4 shows the lines of equi-deflection for the central panel. The approximate location of the lines of inflection for imaginary beams radiating from the column center have been plotted in Fig. 5.

Many interesting conclusions can be drawn from the deflection maps. They show at once the general nature and intensity of the

stresses existing in the plate. They show beyond doubt that tension exists in the upper face of the slab at right angles to the lines joining the columns and forming the sides of the panel, being a maximum at the columns and decreasing to a minimum at the mid-point, although even this minimum value of the tension in the top face is approximately equal to the tension in the bottom face at the center of the panel. Inspection of Fig. 6 will make this more clear. The top curve is a section along the side of a panel

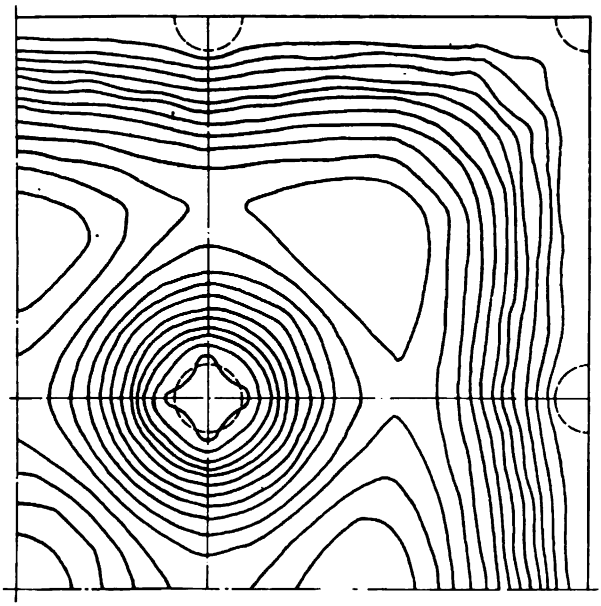

FIG. 3.—CONTOUR MAP OF ONE-FOURTH OF MODEL.

and the lower curve a section through the middle of the panel parallel to one side. From the radii of curvature of these curves, it will be seen that the maximum stress at the side occurs over the column, while the section through the middle shows that the stress is of practically equal intensity at the center of the panel and at the mid-point of the panel edge. Tension in the top surface at the edge of the panel has never been provided for in the reinforcing of flat slabs, although the need for it has been clearly pointed out time and again by the formation of cracks in the

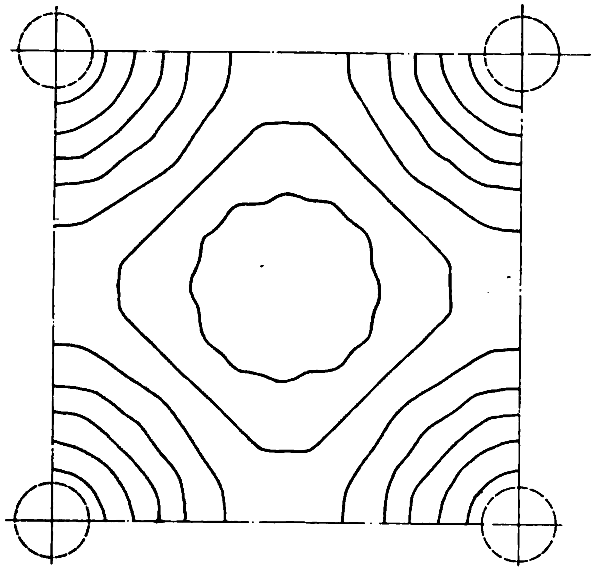

FIG. 4.—CONTOUR MAP OF INTERIOR PANEL.

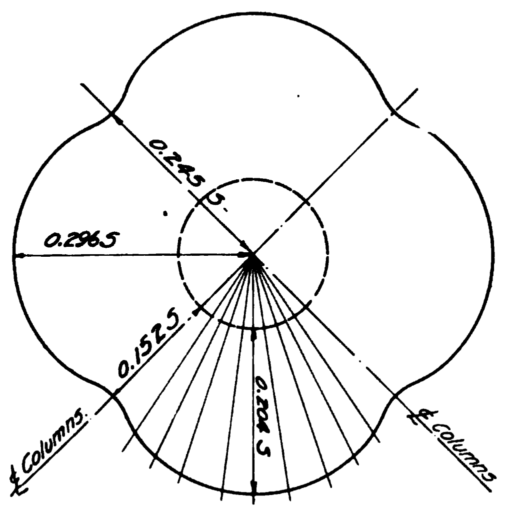

FIG. 5.—LINE OF INFLECTION FOR IMAGINARY RADIAL BEAMS.

concrete. These cracks, which run from column to column along the sides of the panel, have formed in nearly every multiple panel load-test of a reinforced concrete flat slab which has been made, and have even formed in several floors under their dead weight only. It was hoped that the radii of curvature of the elastic surface could be accurately measured and the bending moments deduced from them. It was found, however, that results so obtained were too indefinite to be considered reliable.

It was evident that to obtain exact information as to the distribution and magnitude of stresses existing in the plate, it

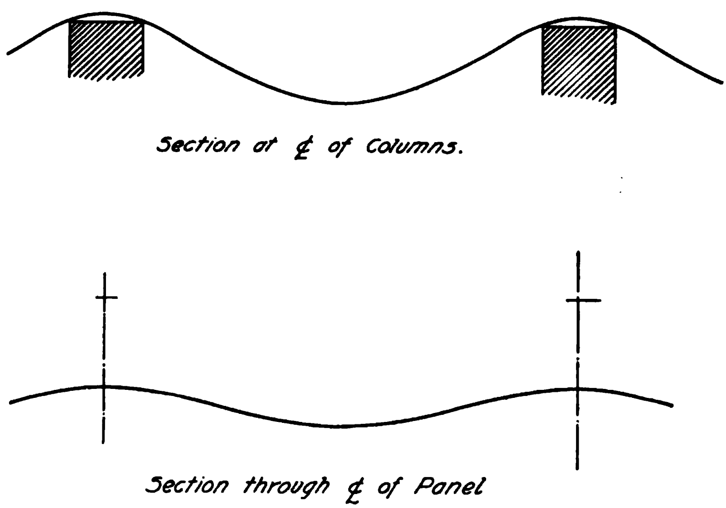

FIG. 6.—SECTIONS AT EDGE AND CENTER OF INTERIOR PANEL.

would be necessary to measure the actual elastic deformation of the material caused by these stresses. This was attempted in many ways; by measuring distortion of squares ruled on the slab; by measuring cracks formed in plaster of paris coatings and lines, and by various forms of extensometers. After trying many devices, it was decided to use a microscope fitted with an ocular micrometer and to measure with it the deformations occurring over a gauge length 0.5 in. long. The ocular micrometer is a disc of glass, engraved with a fine scale, which is inserted in the eye piece. The disc lies in the plane of the image formed so that the engraved lines appear to be upon the image itself, and its length

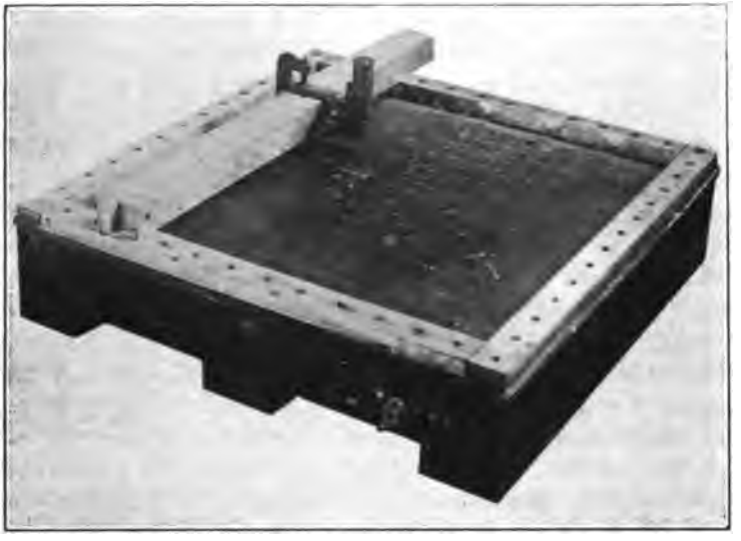

FIG. 7.—MODEL AND EXTENSOMETERS.

can be easily read. The absolute value of one division of the scale depends upon the magnification used, and was approximately 0.00048 in. in these experiments. The total field was about one-fortieth of an inch.

Several devices were tried for bringing the deformations occurring over the gauge length into the field of the micrometer. At first a strip of heavy tinfoil was fastened at each end of the gauge length by passing a fine needle through it into the rubber. The pointed ends of these strips nearly touched at the middle of the gauge length and the distance between them was measured under no load, and again under the various loads employed. This arrangement of tinfoil strips was not satisfactory, as readings taken with it were not consistent with each other, caused probably by slight play of the strips around the needles, or, perhaps, by the strips themselves buckling as the gauge length shortened.

After many trials, the following method was adopted and found entirely satisfactory. A small piece of very fine needle was placed point up in the rubber at one end of the gauge length and at the other end was placed, point down, a piece of needle with the exposed end ground to a triangular shape and highly polished. A small strip of brass was ground to a knife edge at one end, and at the other a small conical depression was made. The point of the first needle entered this depression, and the knife edge was very close to the triangular end of the other needle. The distance from the end of the strip to one point of the triangle was read very easily, and the arrangement gave perfect satisfaction. Fig. 7 shows a general and a detail view of the extensometers in place.

In this first series, these extensometers were arranged on lines radiating from the columns so that the readings both along the lines and perpendicular to them were obtained. Full results of these readings are not given, because it was found that the deformation at a given point increased more rapidly than the loads, indicating catenary action or pure tension throughout the cross-section of the plate. If such action existed, the tensile deformations read in the top fibers of the plate would be higher than the true values caused by bending alone and likewise the compressive deformations of the top fibers would be less than they should be. To test for this, short columns were clamped on the

top of the slab directly over those on the under side and the load reversed. Zero readings for the reversed load were taken under a pressure equaling the dead weight of the slab. Fig. 8 shows readings obtained in this way and under direct load, for points along the line forming the side of the panel. The values of the deformation due to bending moment alone are the arithmetical averages of the direct and reversed readings, and the true zero line has been

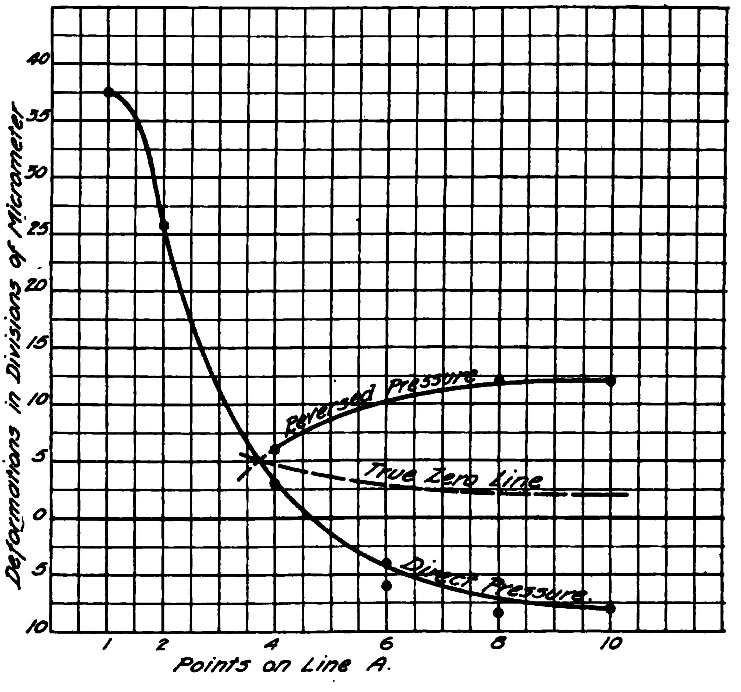

FIG. 8.—DEFORMATIONS UNDER DIRECT AND REVERSED LOADS.

plotted showing the amount of catenary action existing. This method of load reversal was too tedious to be adopted and it was decided to use a thicker plate in the next series of experiments.

The results obtained in the first series of experiments have been discarded in favor of those in the second series and the first series has been regarded as merely preliminary and as having served its purpose in enabling the many mechanical difficulties to be overcome and in indicating many points to be covered.

In the second series of tests the apparatus used was the same as that employed in the first series, except that the rubber plate was 0.485 in. think. This greater thickness was used because it was thought that the effect of catenary action would be less on a heavy plate than the lighter one used in the first series. The correctness of this assumption was later proved by a series of readings of deformations at various points under direct loads

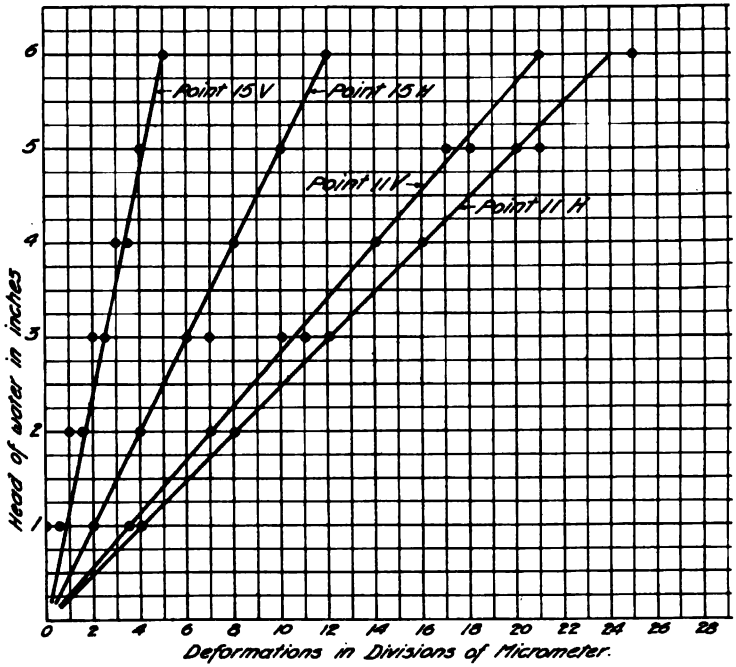

FIG. 9.—PROPORTIONALITY OF STRESS TO LOAD FOR SECOND MODEL.

equivalent to from 1 to 6 in. of water which showed proportionality of deformation to load. The results of these readings are plotted in Fig. 9. The lack of catenary action was also confirmed by reversing the load and obtaining curves identical to those obtained under direct load.

Deflection readings were taken at points over the interior panel spaced one-tenth of the span apart. Fig. 10 is a plan of the interior panel upon which have been plotted contour lines

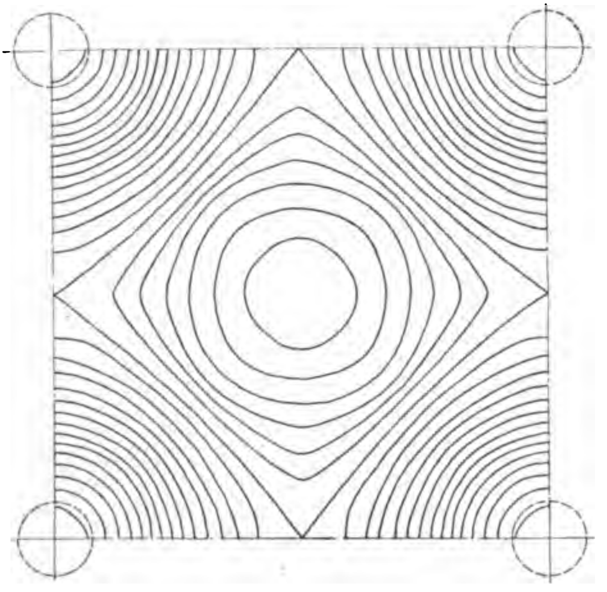

FIG. 10.—CONTOUR MAP OF INTERIOR PANEL.

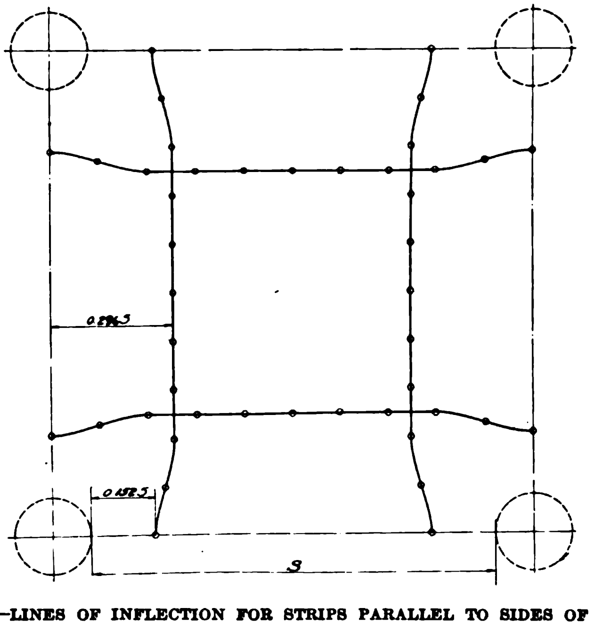

FIG. 11.—LINES OF INFLECTION FOR STRIPS PARALLEL TO SIDES OF PANEL.

representing elastic deflections of 0.01 in. caused by a load equivalent to 5 in. of water. Sections of the surface so defined were taken parallel to the sides of the panel and the points of inflection plotted in Fig. 11. The lines joining these are lines of inflection for imaginary beams or strips parallel to the sides of the panel. These lines agree very well with the results obtained later from deformation readings.

Fig. 12 is an exaggerated sketch of the elastic surface upon which have been marked coefficients for deflection. To apply these to concrete flat slabs reinforced parallel to the sides of the panels a strip of unit width is assumed along the edge of the panel,

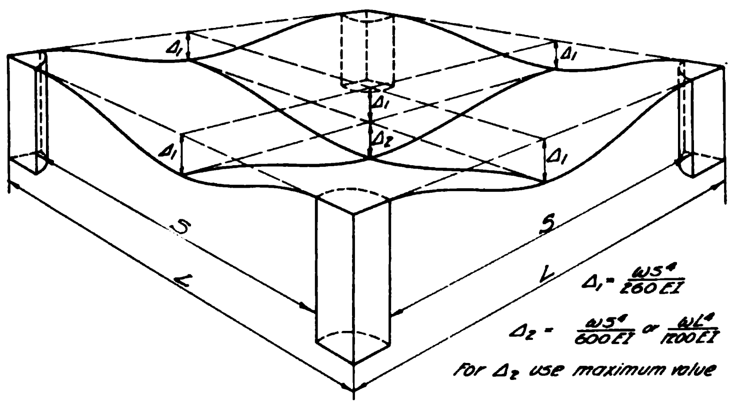

FIG. 12.—DIAGRAM OF ELASTIC SURFACE AND DEFLECTION COEFFICIENTS

and another at right angles to this along the center line of the panel. Knowing the total load per square foot on the panel, as well as the effective depths and steel percentages of each strip, it is an easy matter to obtain the deflections. Instead of working out the values of the moment of inertia of the strips one may follow the method outlined by Mr. Eli White in the *Engineering Record* of November 9, 1907, and elaborated by Mr. G. F. Dodge in his *Diagrams for Designing Reinforced Concrete Structures.* To simplify the computations the strips may first be treated as simply supported beams, and the deflections so obtained multiplied by the ratio of the deflection coefficients.

The results of the first series of tests indicated that tensile reinforcement would have to be provided in the top of the slab

at right angles to the lines forming the sides of the panel, and it was decided that a two-way system of reinforcement would be most adequate, both in providing this reinforcement and in taking care of the stresses existing in other parts of the construction, and consequently the arrangement of the extensometers was as shown in Fig. 13. It will be seen from this figure that deforma-

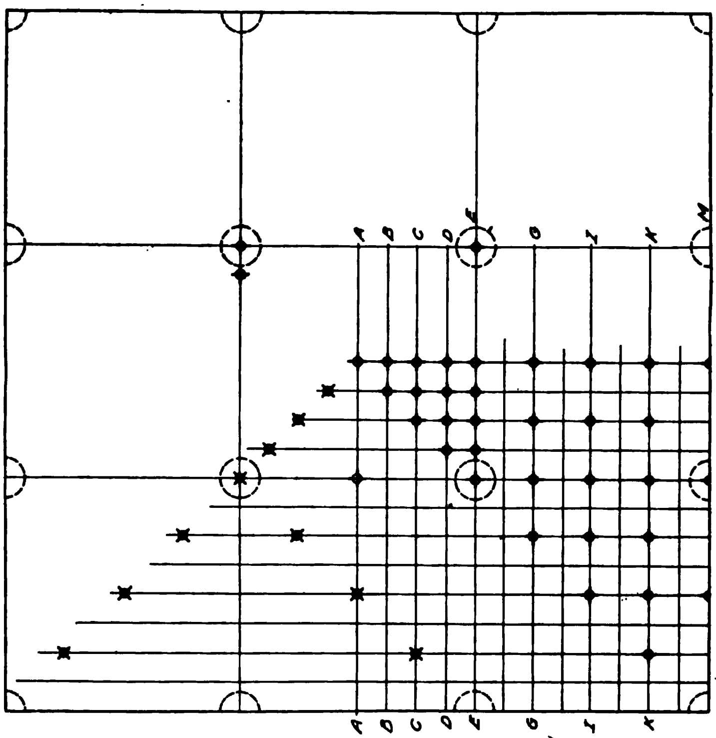

FIG. 13.—ARRANGEMENT OF EXTENSOMETERS

tions were read parallel to each side of the panel at points spaced one-eighth of the span apart, and so distributed that deformations in other portions of the plate were by the symmetry of the construction obtained from readings on these.

It was found that a load equivalent to 5 in. of water gave deformations large enough to be easily read, and this load was therefore adopted. The extensometers at all the points were

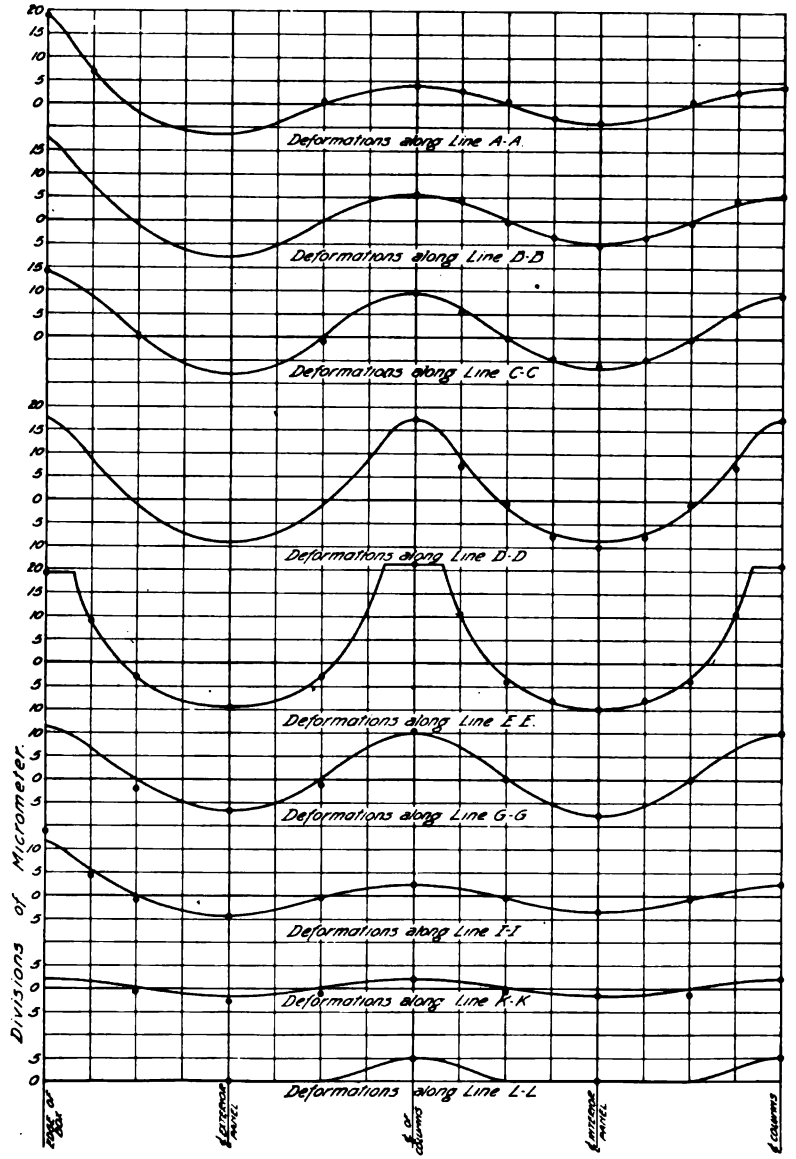

FIG. 14.—MOMENT CURVES.

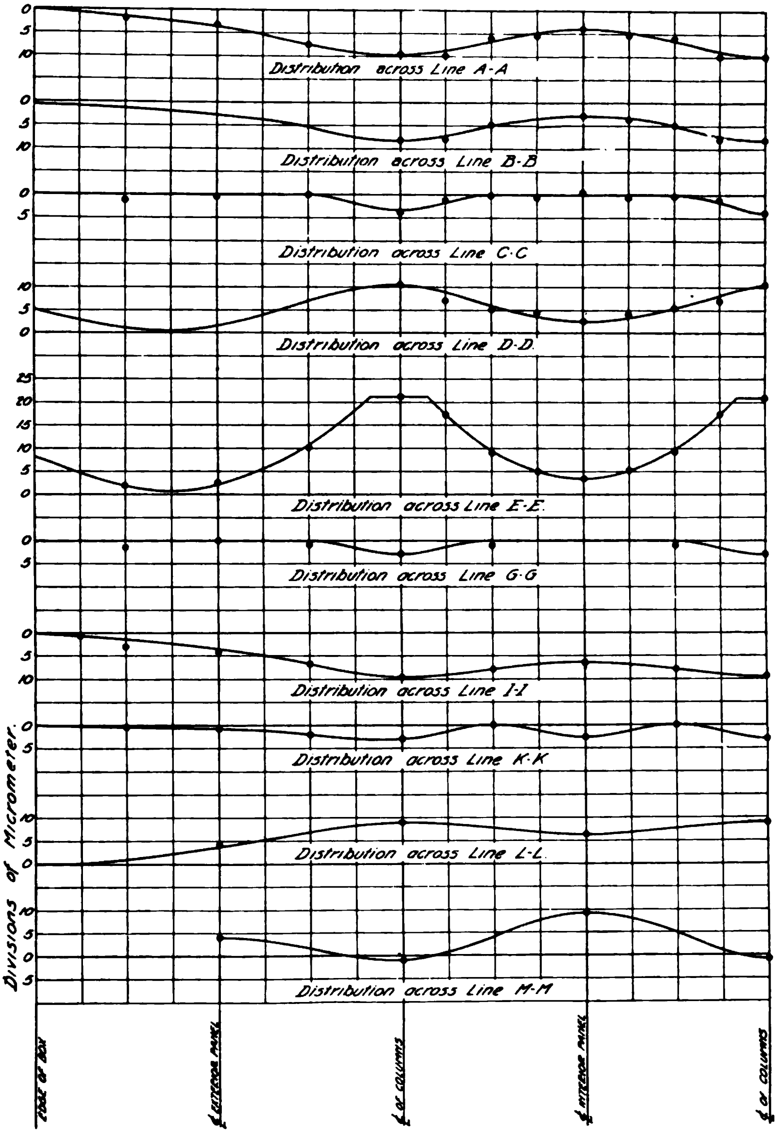

FIG. 15.—DISTRIBUTION CURVES.

read under no load, and again, under load; the differences giving the elastic deformations caused by the load. The load was applied and released time and again, as many as forty readings being taken on some of the critical points by each of two observers. The readings obtained at any given point agreed with one another to within a very few divisions of the micrometer. Table I gives the averages of all the readings taken. It will be seen upon inspection of Fig. 13 that two sets of curves may be plotted from these readings. One set will show moment diagrams for imaginary strips, the axes of which lie along the zero lines of the curves. Curves of the other set show the distribution of moment among various strips of this kind. Since a two-way system of reinforcement was to be used, it was decided to design each set of bars independently. For square panels the reinforcement would, of course, be the same in each direction.

TABLE I.—AVERAGES OF DEFORMATION READINGS.

Point. / Line.	Edge of Box.	$\frac{1}{8}$ Pt.	$\frac{1}{4}$ Pt.	C. L. of Exterior Panel.	$\frac{1}{4}$ Pt.	Line of Cols.	$\frac{1}{8}$ Pt.	$\frac{1}{4}$ Pt.	$\frac{3}{8}$ Pt.	C. L. of Interior Panel.
A	18.8	6.5	−2.25	−1.83	0.5	3.82	2.67	0.5	−3.0	−4.03
B						5.5	4.25	−0.5	−3.75	−5.5
C			0	−5.0	−1.0	9.33	5.33	−0.25	−4.75	−6.0
D						17.33	7.0	−1.0	−8.0	−10.0
E	9.17	9.0	−3.0	−9.5	−3.0	21.25	10.61	−4.0	−8.0	−9.66
G			−2.0	−6.5	−1.17	10.33		0		−7.5
I	14.0	4.5	−0.83	−4.1	−0.25	2.54		−0.5		−3.33
K			−0.67	−3.0	−1.67	2.0		−1.33		−2.0
L					−0.5		5.0			−0.75

Fig. 14 shows the moment curves for strips parallel to one side of the panels, while Fig. 15 shows distribution curves for these same strips. These two sets of curves can be combined to form a surface showing both moments and distribution for one set of strips. A photograph of this surface for four panels in one corner of the model is shown in Fig. 16.

Fig. 17 is a sketch showing the surface for the interior panel. In this figure the ordinates from the plane *A-B-C* to the surface are proportional to the deformations of the top fiber of the plate parallel to side *B-C* at the point at which the ordinate intersects the surface. Sections parallel to *B-C* will be the moment dia-

grams similar to those in Fig. 14, and sections parallel to *A-B* will be distribution curves similar to those shown in Fig. 15.

In Figs. 14 and 15 the ordinates to the curves are shown in terms of divisions of the micrometer. In order to co-ordinate divisions of the micrometer with bending moment, a strip of the rubber was cut from the plate, and the stress strain diagram shown in Fig. 18 obtained. Both the axial and lateral deformations of the strip were measured, from which the modulus of elasticity was found to be about 1,000 and Poisson's ratio about .44; one

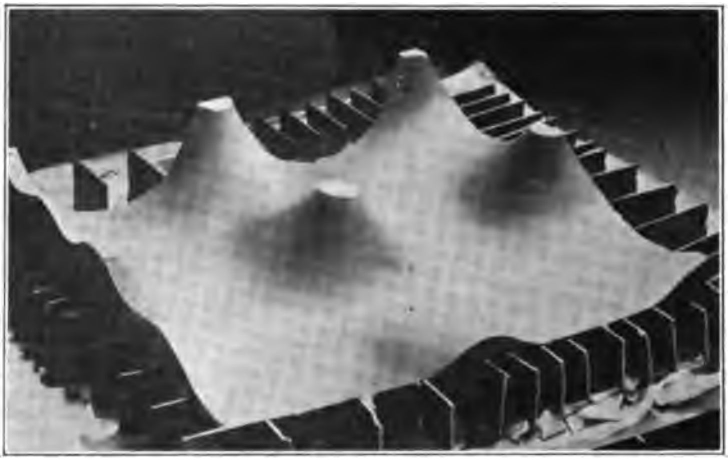

FIG. 16.—MOMENT SURFACE FORMED BY COMBINING MOMENT CURVES AND DISTRIBUTION CURVES.

division of the micrometer corresponding to a unit elongation of 0.001.

The value of one division of the micrometer in terms of bending moment was, however, obtained directly in the following manner:

A strip of rubber cut from the plate was supported over two knife edges, and loaded as shown in Fig. 19. An extensometer exactly like those used on the plate was placed on top of the strip between the supports. Known weights were applied, and the accompanying deformations read. Fig. 19 shows deformations so obtained, plotted against bending moments in in.-lb. per inch

width of strip. From this curve the moment equivalent of any number of divisions of the micrometer can be obtained. The unit load on the panel being known, together with the span, what might be termed the "gross bending moment" or $M = wS^2$, may be obtained. In this case the clear span S was 6.75 in. and the load equivalent to 5 in. of water, or .1808 lb. per sq. in., so that $M = wS^2 = 8.2377$ in.-lb. If the net moment at any point be $M = wS^2/z$, the value of the z may be obtained from the moment curves in Fig. 14 and the calibration curve in Fig. 19. With S

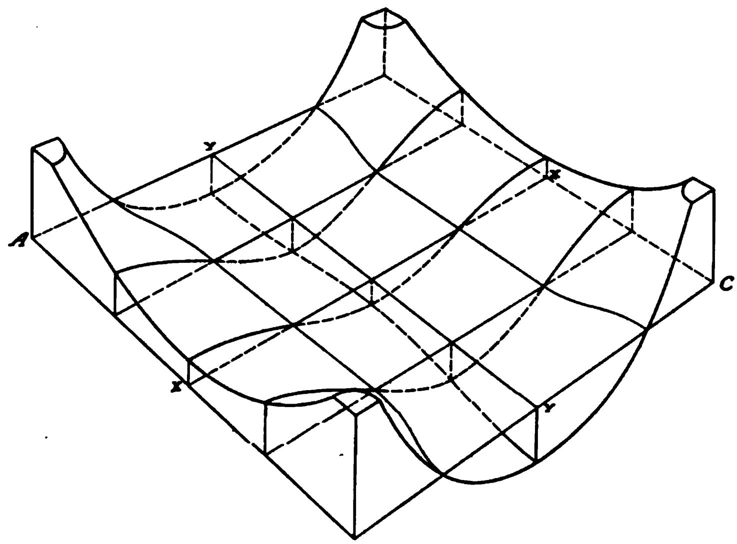

FIG. 17.—DIAGRAM OF MOMENT SURFACE FOR CONTINUOUS FLAT PLATE SUPPORTED AT CORNERS.

equal to the clear span from edge to edge of column heads, $z = 215/D$, where D is the ordinate to the moment curves in divisions of the micrometer.

On Fig. 20 the moment expressions obtained in the above manner have been marked at the critical points.

Having now obtained an empirical analysis of a homogeneous plate it will be applied to flat slabs of reinforced concrete. This may be done without hesitation because the whole history of the development of reinforced concrete has been along such lines. It was found that once having solved the problem of internal

stress distribution in a concrete beam, there could be safely used the equations for external moments applying to homogeneous beams, and so on.

In this empirical analysis the components in two directions of the stresses existing in a homogeneous plate have been measured and reinforcement will be supplied in the concrete flat slab to resist these components where they show tensile stresses. In

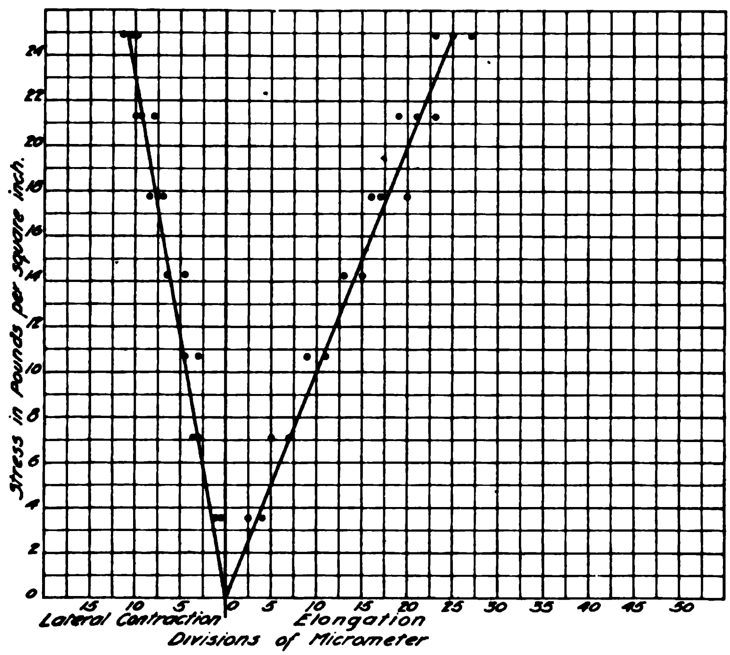

FIG. 18.—STRIP FROM PLATE LOADED FOR MOMENT CALIBRATION OF EXTENSOMETERS.

laying out the reinforcement a departure is made from the old diagonal or four-way system and the two-way system, Fig. 21, employed.

One of the greatest advantages of the two-way system is the location of tensile reinforcement in the top of the slab between the columns, which tends to prevent the formation of cracks extending from column to column and stiffens the whole structure, giving all the advantages of continuous construction.

Another point is that the tensile reinforcement over the columns is in but two layers and hence the effective depth at this critical point is much greater than would be possible with the systems in which four or more layers are used.

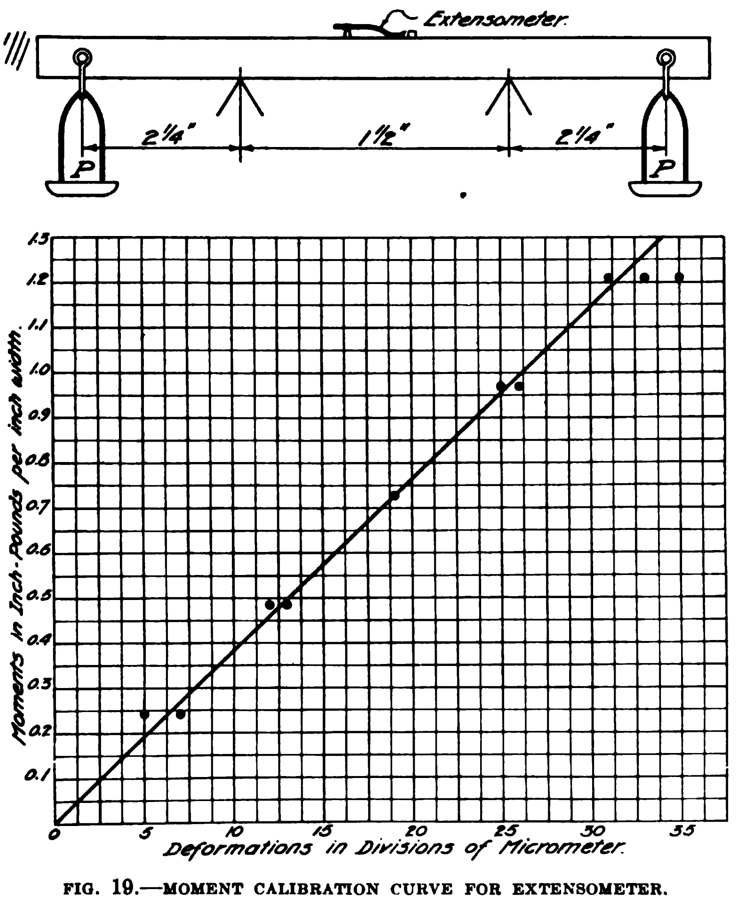

FIG. 19.—MOMENT CALIBRATION CURVE FOR EXTENSOMETER.

The ideal reinforcement would be such that its area at any point be directly proportional to the ordinate of the moment surface shown in Fig. 17, but such reinforcement is obviously impossible in practice. The panel is, therefore, assumed to be divided into ten strips or imaginary beams, five parallel to each

side, and such moments assumed for their design as will give reinforcement areas conforming most closely to the theoretical values.

The widths of these imaginary beams together with the moment factors applying to each are shown in Fig. 22. To avoid confusion in the drawing, the widths and moments for but

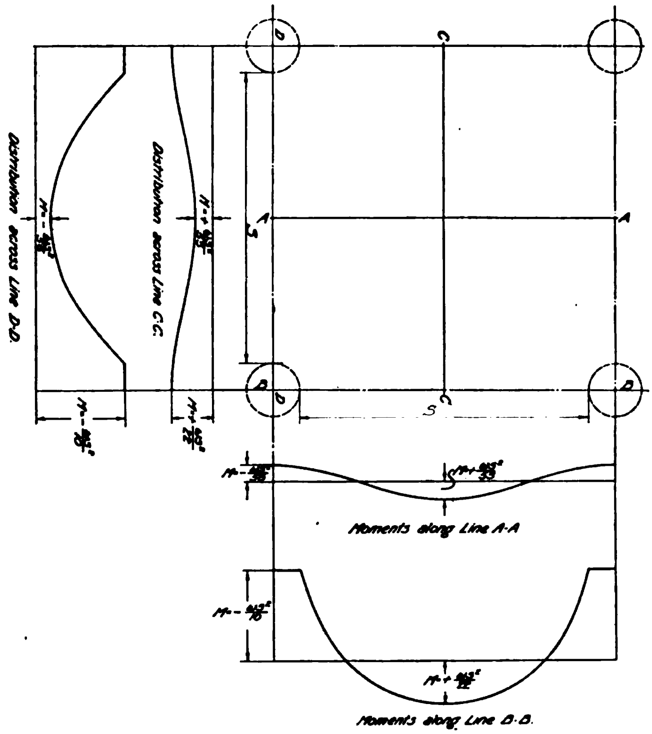

FIG. 20.—MOMENTS PER UNIT WIDTH DERIVED FROM EXPERIMENTS.

one set of strips are shown, those for the other set being the same but turned through ninety degrees. The numbers in the circles on the diagrams are moment coefficients entering into the denominator of the moment equation. Thus, the moment at the center of the panel in the middle strip or beam is $M = wS^2/40$, where w is the total unit live and dead load on the panel in lb. per sq. ft., s is the clear span in feet, and the 40 is the moment coefficient

FIG. 21.—REINFORCEMENT IN PLACE.

shown in the circle at this point. M is the bending moment in ft.-lb. per ft. of width, and is positive, requiring reinforcement in the bottom of the slab, as indicated by the fact that the arrow showing the direction of the reinforcement is dashed. The moments of other points of the panel are obtained in the same manner. The sections at the bottom of Fig. 22 show how the theoretical distribution curves are replaced by more practical stepped lines. In making this change an excess of reinforcement has been introduced at the center of the strips to provide for single panel concentrations of loading.

Since the maximum stresses in a flat slab occur at the columns, the use of a slab of uniform thickness is wasteful of concrete and adds unnecessary dead load. The unnecessary concrete can be done away with, decreasing the weight and cost of the structure and at the same time adding to its efficiency, if a rectangular cap of concrete is left over the columns, extending to about the fifth point of the span.

The general method of procedure in designing a concrete flat slab in accordance with this method is to first find the areas of reinforcement required in the different bands at the center line of the panel. These areas are supplied by rods of the proper size and spacing, every other one of which is bent up in the top at about the quarter point and extends over into the adjacent panels, while the remainder are straight and extend about 6 in. past the edge of the panel. This arrangement gives the necessary amounts of reinforcement in both the top and bottom of the slab, except in the areas over the columns where the necessary extra reinforcement in the top is supplied in the form of short, straight bars, remembering that at these points the effective depth of the slab is increased. To avoid bunching the bars in the top of the slab, a bent bar in one panel should be in line with a straight bar from the next panel.

The line of inflection may be safely taken at the fifth point of the clear span, or at the quarter point of the center to center spacing of the columns, using the higher value in every case. The point of bend in the bent bars, and also the length of the extra short rods in the top, will be governed by this dimension.

In the end panels the area of the reinforcement perpendicular to the wall should be increased by the usual 20 per cent. The

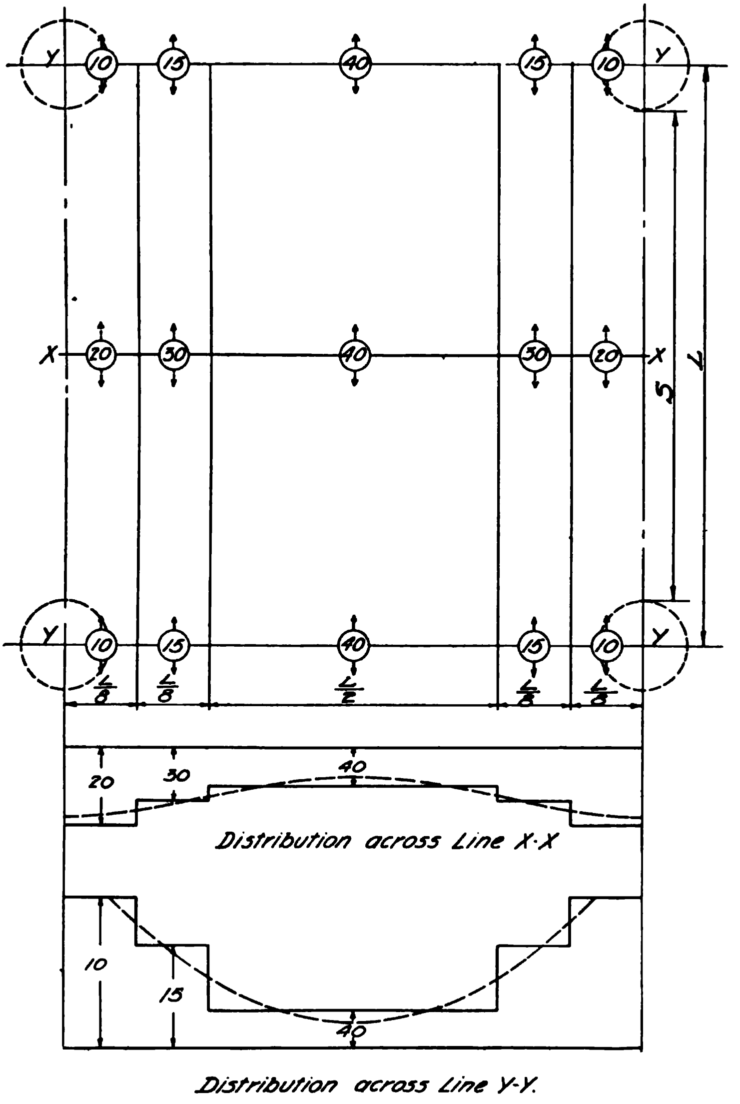

FIG. 22.—MOMENTS PER UNIT WIDTH USED IN DESIGN.

easiest way to do this is to use round bars in the interior panels, and square bars of the same nominal size and spacing in the end panels. In order to facilitate the placing of the reinforcement, it will be found necessary to provide light bars, preferably $\frac{3}{8}$ in. round, along the sides of the panels, so that they may be used to support the bent bars. Of course, these spacing bars may be figured as effective in tension over the top of the column, and thus replace some of the short bars used there.

Table II gives recommended dimensions for flat slab floors to suit various live loads and panel sizes. The table is based on a maximum theoretical concrete stress of 750 lb. per sq. in. The slab thicknesses are such that the deflections under a superimposed test load equal to once the dead load plus twice the live load will not exceed one five-hundredth of the span for a theoretical working stress in the reinforcement of 18,000 lb. per sq. in. Tension in the concrete, arch action, etc., all tend to reduce the actual stresses below the theoretical values, but no building code will permit these factors to enter into the design of concrete structures.

Flat slab designs showing smaller column head diameters, or thinner slabs over the supports, than those given in the table should be carefully checked for shear at the edge of the column heads.

To illustrate the application of this new method for the computation of flat slabs, the detailed design of a typical panel will be given. To enable comparison of the results with those obtained by other methods described and tabulated by Mr. Angus B. McMillan in a paper* before the Association, a panel 20 ft. square will be designed for a live load of 200 lb. per sq. ft., using a steel stress of 16,000 lb. per sq. in. and a ratio of the moduli of steel and concrete of 15.

In Table II under the above span and loading a slab thickness of 8 in. is given, with a 2 in. cap 8 ft. square, and a column head 50 in. in diameter. The average weight of the slab, including the cap, is 104 lb. per sq. ft.

The clear span from edge to edge of column heads is 15.83 ft., and the "gross bending moment" or wS^2 is $12 \times 304 \times 15.83^2$ or 914,150 in.-lb. per ft. of width. Referring to Fig. 22, it will be

* *Proceedings*, Vol. VI, p. 248.—ED.

TABLE II.—RECOMMENDED DIMENSIONS FOR VARIOUS LOADS AND COLUMN SPACING.

L ft.	C ft.	C in.	40 Lb.				150 Lb.				200 Lb.				300 Lb.				400 Lb.				500 Lb.			
			T in.	H in.	D in.	Wt. lb.	T in.	H in.	D in.	Wt. lb.	T in.	H in.	D in.	Wt. lb.	T in.	H in.	D in.	Wt. lb.	T in.	H in.	D in.	Wt. lb.	T in.	H in.	D in.	Wt. lb.
15	6	0	5	7	16	67	6	8	30	79	6	8	36	79	7	9	36	92	7½	11½	36	102	8½	12½	38	114
16	6	6	5	7	18	67	6	8	34	79	6½	8½	38	86	7½	9½	42	98	8	12	40	108	9	13	44	121
17	6	9	5	7	20	67	6	8	38	79	7	9	42	92	8	11	44	106	8½	12½	46	114	9½	13½	48	127
18	7	3	5	7	22	67	6½	8½	40	86	7	9	46	92	8½	11½	46	112	9	13	50	121	10	14	50	133
19	7	9	5	7	26	67	7	9	44	92	7½	9½	48	98	8½	11½	48	112	9½	13½	52	127	10½	15½	52	142
20	8	0	5½	7½	28	73	7½	9½	46	98	8	10	50	104	9	13	50	120	10	14	54	133	11	16	56	148
21	8	6	5½	7½	30	73	8	10	50	104	8	10	52	104	9½	13½	52	127	10½	14½	56	139	11½	16½	60	154
22	8	9	6	8	32	79	8	10	54	104	8½	11½	54	112	10	14	54	133	11	16	58	147	12	18	60	162
23	9	3	6½	8½	36	86	8½	10½	56	111	9	12	56	119	10½	14½	56	140	11½	16½	60	154	12½	18½	64	168
24	9	9	7	9	38	92	9	11	58	117	9½	12½	58	125	10½	15½	58	142	12	18	62	162	13	19	68	175
25	10	0	8	10	40	104	9½	11½	60	123	10	13	60	131	11	16	60	148	12½	18½	64	165	14	20	70	187

seen that the net positive moment for the central strip of the slab is 1/40 of the gross moment, or 22,850 in.-lb. per ft.

To enable the comparison with Mr. McMillan's table, the effective depth of the slab will be taken as 7 in., so that the area of reinforcement per foot width of this strip—

$$A_{40}=\frac{22,850}{7\times.86\times16,000}, \text{ or } 0.237 \text{ sq. in.}$$

This will be supplied by using $\frac{5}{8}$-in. corrugated round bars spaced 15 in. on centers. The negative moment for this strip is of the same magnitude as the positive, so that it can be properly taken care of by bending up every other bar in this strip and in the corresponding strips of the adjacent panels.

At the center of the next strip, there is a positive moment equal to 1/30 of the gross moment, or 30,470 in.-lb., requiring an area of reinforcement per foot—$A_{30}=0.316$ or $\frac{5}{8}$-in. rounds, $11\frac{1}{2}$ in. on centers.

Over the edge of the panel the negative moment for this strip is 1/15 of the gross moment, or 60,940 in.-lb.—

$$A_{15}=\frac{60,940}{9\times.86\times16,000}, \text{ or } 0.492 \text{ sq. in. per ft.}$$

If every other bar from the bottom of this band is bent up, there are 0.316 sq. in. per ft. and the remainder, 0.172, can be supplied in the form of short, straight bars, using $\frac{5}{8}$-in. rounds spaced 21 in. on centers.

For the strip or band at the edge of the panel the net positive moment is 1/20 of the gross moment, or 45,710 in.-lb., calling for $A_{20}=0.474$ sq. in. per ft., or $\frac{5}{8}$-in. rounds spaced 7.5 in. on centers. At the ends of this strip over the column head the net negative moment is 1/10 of the gross moment, or 91,415 in.-lb., requiring $A_{10}=0.738$ sq. in. per ft. The bent bars will supply 0.474 sq. in. per ft., so that the remaining 0.264 sq. in. will be taken care of by short straight bars, using $\frac{5}{8}$ in. rounds spaced 13.5 in. on centers.

The length of the short straight bars will be such that they will reach the fifth point of the clear span, so that they will be 10 ft. 6 in. long for this panel.

Having determined the size and spacing of the bars they can be laid out on a drawing, or the number required estimated in the following manner:

The balanced average of the moment factors in Fig. 22 for the positive moments at the center of the panel is 30, so the net moment is 30,470 in.-lb. per ft., or in the 20 ft. width of panel, 609,400 in-lb. To resist this moment will require 6.33 sq. in. of reinforcement, or twenty-one $\frac{5}{8}$ in. round bars. The average area of the short top bars required is 0.218 sq. in. per ft., and the total width in which they must be supplied is 10 ft., so that 2.18 sq. in., or say seven $\frac{5}{8}$-in. round bars are required.

This gives the total number of bars in one direction; in the whole panel there are 21 straight bars 21 ft. long, 21 bent bars 31 ft. long, and 14 straight bars 10 ft. 6 in. long, all $\frac{5}{8}$-in. round, or a total of 1300 lb. of reinforcement per panel. Table III gives a

TABLE III.—QUANTITY OF REINFORCEMENT REQUIRED PER PANEL UNDER VARIOUS DESIGNS WITH A UNIT REINFORCEMENT STRESS OF 16,000 LB. PER SQ. IN.

Method.	Thickness of Slab, in.	Pounds of Reinforcement.
Cantilever	8	2,189
Turneaure and Maurer	12	1,931
Grashof	8	784
Mensch	8	2,120
Turner	8	549
McMillan	8	1,084
Brayton	8½	1,900
Trelease	8	1,300

comparison of the quantity of reinforcement required under various designs.

Fig. 23 shows the arrangement of the reinforcement in this typical panel. To avoid confusion in the drawing, the bars from the adjoining panels are not shown. In designing this slab, it has been assumed that the slab is fixed at the edge of the column head, as it is the general practice to flare the head at a small angle, making it very rigid. The head is not counted on to resist bending moment, and is flared merely to take care of the shear and to somewhat reduce the clear span.

To compute the deflection of the typical panel designed, the deflection of a strip of unit width at the side of the panel will be computed and also of a unit strip at the middle of the panel, and the total deflection of the plate taken as the sum of these. It will

first be assumed that these strips are freely supported, and the deflections transformed later by the ratio of the deflection coefficients given in Fig. 12 to those for simply supported beams. The

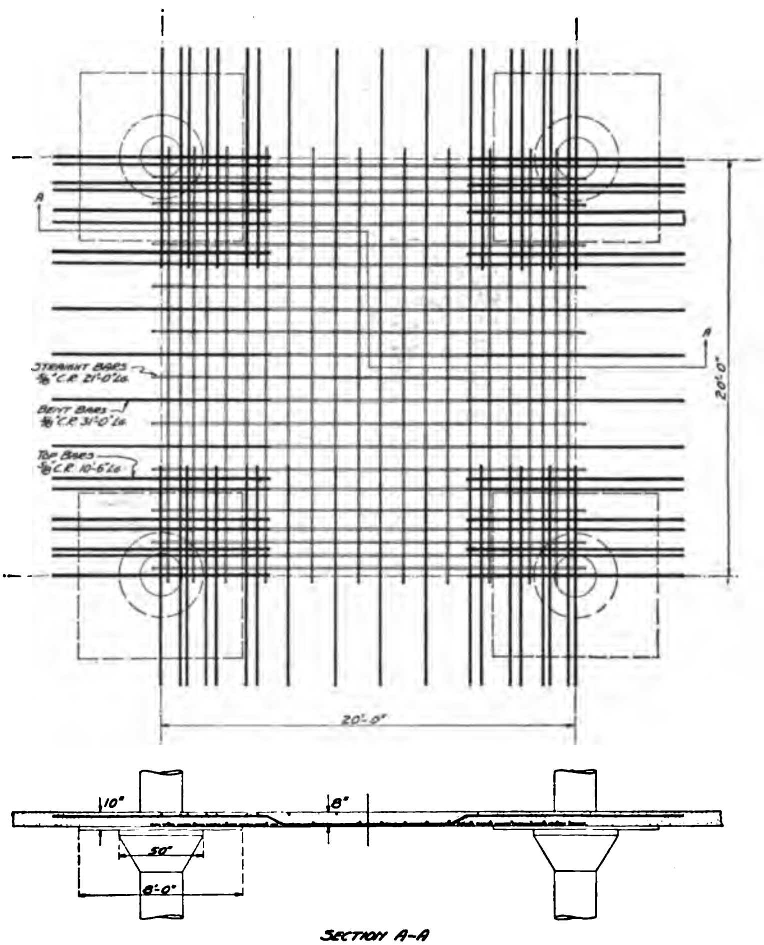

FIG. 23.—ARRANGEMENT OF REINFORCEMENT IN TYPICAL PANEL.

computations may be very quickly made by using the graphical solution of Mr. White's method published in *Dodge's Diagrams*.

In this way the deflection at the working reinforcement

stress of 16,000 lb. per sq. in. is found to be 0.15 in., or 1/1600 of the panel side. For higher stresses the deflection would, of course, be greater.

The unit stresses which have been employed for this panel are very conservative, and could be safely increased to 18,000 lb. on the reinforcement and 750 lb. on the concrete. In this particular panel the maximum concrete stress is at the column head and is only 600 lb. per sq. in., while at the center of the side band eth concrete stress is about 550 lb. With the 50-in. column head used the shear over *jd* is about 100 lb. per sq. in. and as this is

FIG. 24.—TEST LOAD ON PANEL IN ST. LOUIS.

punching shear, such as in footings, a higher value could be used if desired.

Floors designed in accordance with the methods outlined in this paper have been built in several cities, and a few have been thoroughly tested. In these tests, both reinforcement and concrete stresses have been measured by extensometers. The floors tested were designed for reinforcement stresses of 20,000 lb. per sq. in. with the exception of the building in Minneapolis, where 18,000 lb. was used. These theoretical stresses were, of course, not realized in the tests as tension in the concrete, arch action, etc., were not taken into account in the designs.

Fig. 24 shows a panel in a factory in St. Louis under a test

load of 400 lb. per sq. ft. The panel was 20 ft. $\frac{1}{2}$ in. by 22 ft., designed for a safe live load of 150 lb., with an 8-in. slab. The slab was inverted in this case, giving a perfectly flat ceiling. Two inch caps were to be cast on the top and hidden in the cinder fill, and the drawings showed the bars bent up into them. As actually constructed, the reinforcement in this panel had an effective depth over the column, the critical point of the structure, of but 60 per cent of that shown on the drawings. Even under these

FIG. 25.—ADDITION TO THE FORD MOTOR COMPANY'S FACTORY, DETROIT, MICH.

adverse conditions, the maximum reinforcement stress over the column head under the full test load was but 17,000 lb., and the concrete stress at this point 950 lb. The reduction of the effective depth over the column caused a large reduction of the moment of inertia at that point, giving conditions approaching those in a freely supported panel, as the stresses and deflections clearly indicate. The deflection increased from the normal amount for a fixed panel to 0.68 in. under full test load, somewhat less than the normal amount for a free span. The average reinforcement stress

at the middle of the panel was also increased in like manner to 25,000 lb. per sq. in.

Perhaps the most severe test of a floor was one made in Minneapolis in which four panels of a warehouse floor were loaded.* The building as laid out was not well adapted to flat slab floor construction, because of the number and shape of the panels. There was one row of columns through the building, dividing the floor into panels 13 ft. 5 in. by 19 ft. 11 in.; with the short side resting on brick bearing walls. An 8-in. slab was used for the 200 lb. live load required. Four panels of the floor were loaded with cement piled to prevent arching, and very complete readings of stresses and deflections were made. Under the maximum superimposed load of 400 lb. per sq. ft. the deflection at the middle of the panels averaged 0.39 in., a very good figure remembering that the panels were free at one end. The reinforcement stress over the column head was 23,000 lb., and at the mid-point of the span 13,500 lb. The concrete stress had a maximum value at the column head of 1050 lb. per sq. in. These stresses show the design to be conservative.

Fig. 25 is a view of a factory in Detroit having floors and roof of this type of flat slab. The panels are 25 x 20 ft. and the floors were designed for 150 lb. per sq. ft., using an 8-in. slab, 2-in. cap and 48-in. column head. One panel of the floor was loaded with gravel to 300 lb. per sq. ft. Under this load, the deflection was found to be 0.60 in. The reinforcement stress over the column head was 13,500 lb. per sq. in. and at the center of the slab 13,850 lb., showing a well-balanced design.

These tests seem to show quite conclusively that the method of design outlined can be used for almost any condition of span and loading with confidence as to the resulting strength of the structure.

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* For complete data of test see Report of Committee on Reinforced Concrete and Building Laws, p. 108.—ED.

DISCUSSION.

Mr. Alfred E. Lindau.—The object of the test was to gain some knowledge of the action of an elastic slab, supported on points, under a uniform load. At the time the tests were undertaken it was not expected to obtain a great deal of information that could be used in actual designing of concrete structures, but rather some idea of the nature of the deformations; by plotting the deformations obtain the deflection of the surface and perhaps determine points of inflection, rate of change of curvature at the various points and perhaps some idea of the relative deformation for various points of the slab. Mr. Lindau.

Many suggestions were made as to the material to be used in making this preliminary investigation. It was suggested that a steel plate be polished and by some mirror apparatus obtain those deformations. Hard rubber was also suggested, plaster plates were tried as well as a cement coated wire screen—all of which gave some valuable information but did not give the information desired and the results would not have been of any particular interest.

Mr. Arthur N. Talbot.—I have been very much interested in the experiments made by Mr. Trelease. They show the distribution of bending moment in a homogeneous slab of rubber in a way which could not be obtained by mere computation. The negative moments at and between the supports, the relative values of the positive bending moments at the center of the panels are well brought out and there are many features of the results which will be helpful in the application to design problems. Of course, we have here a material that resists tension all through it, as well as compression and one having a modulus of elasticity much the same in tension as in compression. In the case of ordinary flat slab construction the concrete is in compression on one side and has a reinforcement of steel on the other to take most of the tensile stresses. The conditions in the test must be kept in mind because they are different from those of reinforced concrete. Rubber is a material with a very low modulus of Mr. Talbot.

Mr. Talbot. elasticity, a material that even under small deflections would bring in the question of catenary action, rather more than beam or slab resistance.

I wish to ask whether in the experiments an effort was made to determine the effect on the elasticity or on the deformations occurring when the rubber was stretched or compressed in two directions. This has a bearing on slab action.

Mr. Lindau. MR. LINDAU.—There was an attempt made to obtain such results, but some difficulty was encountered in getting satisfactory compression tests on the strips used. The results on the lateral deformation in tension were quite satisfactory and seemed to give very uniform results, but the compressive test did not show up quite so well. The rubber was only stretched in one direction. The question of obtaining lateral deformation was discussed at considerable length but tests were not carried out, largely on account of lack of time.

Mr. Talbot. MR. TALBOT.—The matter of compressive strength is one of considerable importance in flat slab construction, for with a rather small percentage of reinforcement in each of four directions, or even two directions, the combined calculated stress may run high, unless reinforcement in compression is used, which to my mind is not a very satisfactory arrangement with this type of construction. It may be expected that when concrete is subjected to compressive stresses in two directions, its resisting strength will not be the same as when the pressure is applied in only one direction. It is true—and this has a bearing upon the distribution of stresses between the reinforcement and concrete in such construction—that the amount of compressive deformation resulting from applying a given load is less when the load is applied in two directions than when it is applied in one; that seems to be the result of some experiments which we have made and which agree with tests made elsewhere. Specimens were made up in the form of a cross and a load applied in one direction and also in two directions and measurements taken on the concrete in the cross portion and in the arms of the cross; and, making a comparison, there seems to be less shortening in the concrete where it is stressed in two directions, and of course a greater expansion in the direction at right angles than when the compressive stress is applied in only one direction.

We have attempted to make tests of cubes by applying compression against four faces instead of two. The first tests showed considerably greater strength for loads applied in two directions than in one, but we felt that a large part of this difference was due to the restraint of the loading faces themselves, the friction along the face, which kept the concrete from expanding laterally. We then made other tests by lubricating the faces of the bearing blocks very carefully and the values for compression in two directions were approximately the same as those in one. Mr. Talbot.

I may add that measurements were made on cantilever slabs, slabs composed of what you may think of as the column capital and the part of the floor slab out to the line of inflection, supporting them at this line of inflection, just as the load of the remainder of the panel would be suspended in an analysis of the flat slab—omitting, you will see, the uniform load for the central portion immediately surrounding the column—measurements of the deformations of the reinforcement were made. We find that in the middle line of the band the maximum stress is the greatest at the edge of the column capital; that the stress decreases pretty regularly from there to the edge of this cantilever slab, the line of inflection; that through the middle portion immediately over the column capital it decreases rapidly, although it does not become zero with the ordinary size of bars even directly over the center of the column; that along the edges of the bands the maximum stress in the rods is somewhat further in, nearer the center of the length of the reinforcing bars. The maximum stress in the bar at the middle of the band does not differ far from the maximum stress in the bar at the edge of the band, even though its position differs considerably.

THE PRACTICAL DESIGN OF REINFORCED CONCRETE FLAT SLABS.

By Sanford E. Thompson.*

The purpose of this paper is to present material covering the practical task of designing flat slab floors for reinforced concrete structures. The requisite thickness of slab, amount of reinforcement and size of column head for different loadings and different spans are given in Tables I–IV; and the theories and assumptions involved in the computation are briefly discussed. Values not included in the tables may be worked out from the formula, finding the desired values of C_5 and C_6 from the diagrams.† Curves are given also for the constants used in the design of members with reinforcement in top and bottom, and apply not only to flat slabs, but to any beam or slab reinforced both in compression and tension.

For reinforced concrete buildings, the flat slab, or girderless floor,—as it is sometimes called,—is as cheap, and frequently cheaper, than beam and girder construction. The smooth ceilings with no intersecting beams allow better distribution of the light. The expense and complication of installing sprinkler systems are reduced. The clear headroom for the same story height is increased, or else, on the other hand, the story height may be made less without reducing the effective headroom. This last consideration alone is often important enough to dictate flat slab floors.

With flat slab floors the entire load is supported directly on the columns, which are usually spaced about equally in both directions. The column heads are enlarged so as to give increased resistance in shear and bending at the points where this is most needed. The reinforcing bars run through the slabs over the column heads in four directions, two rectangular and two diagonal.

The simplest way of considering the flat slab is to assume

* Consulting Engineer, Newton Highlands, Mass.

† For an example of flat slab design worked out in detail see Taylor and Thompson's "Concrete, Plain and Reinforced," 2d edition, 1911, pages 487 and 488.

that a portion of the slab extending a certain distance out from the column is a flat, circular plate, similar to a Japanese parasol, but with no slope to its surface. This plate is fixed to the column and is assumed to extend out from it on all sides like a cantilever as far as the line of inflection of the slab, which line,—as in other forms of monolithic construction,—is about one-fifth of the net span away from the support. The rest of the slab may be considered as entirely separate from the flat circular plates but simply supported from their outer edges or circumferences.

This is no new theory but is somewhat similar in effect to that of a uniformly loaded, fixed or continuous beam. To illustrate this in practical fashion, an ordinary beam uniformly loaded and fixed at both ends will be considered. This illustration does not in any way show the methods of determining a bending moment in the flat slab, since, as stated below, the actual bending moment is dependent upon the elastic theory. It does, however, show quite clearly the justification of assuming the slab to be cut through on the line of inflection.

It is known from simple mechanics that the moment at the support of an ordinary fixed or uniformly loaded continuous beam is $Wl/12$* and, at the center, is $Wl/24$. Now, suppose at the points of inflection, which also by mechanics are known to be located at a distance $0.2113l$ from each support, the beam is cut completely through so as to have a cantilever at each end with a simply supported beam between. The bending moment of the cantilever at its support, due to the load upon it, is $0.2113 W\times 0.2113l/2$, and the moment at its support due to the load on the supported beam between cantilevers, is $[1-2\ (0.2113)]2 W\times 0.2113l$. The sum of these two moments is $0.0223 Wl+0.0610\ Wl=0.0833\ Wl$ or $Wl/12$. In other words, while this analysis is not that which can be used for a flat slab, because of the extra strength of the flat slab due to the multiple reinforcement, the division into sections corresponds to our assumption in the flat slab theory. In the same way it might be shown that the center moment of the simple beam supported by the two ordinary cantilever beams is $Wl/24$.

Tests of the flat slab construction at Minneapolis† indicate

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* W = total live plus dead load. l = distance in feet between supports.

† See paper on "A Test of a Flat Slab Floor in a Reinforced Concrete Building," by Arthur R. Lord, *Proceedings*, Vol. VII, page 156.

that the line of inflection of a flat slab floor is substantially the same as in a fixed beam, or about $^1/_5$ the net distance between supports, although, as would be expected, the bending moment is entirely different.

PROBLEM OF DESIGN.

The problem of the design of the flat slab, then, resolves itself into (1) a determination of the proper thickness and reinforcement required at the support for the cantilever circular plate supporting its own load and also the load of the rest of the slab, and (2) a determination of the thickness and reinforcement at the center of the span required for the simply supported section lying between the circular plates.

VARIOUS METHODS OF DESIGN OF SLAB.

Various methods have been advanced for the design of the flat slab. Some are based merely on deflection tests, which give no true basis for computations; others compute the reinforcement carefully at the center of the slab, which is not the critical part; others consider the construction to consist of beams between columns with a slab between, thus obtaining ultra-conservative results; while a plan still more common is to take the moment at the supports arbitrarily without regard to the size of the column head. The shear or diagonal tension near the column head is frequently disregarded altogether.

SHEAR AT THE SUPPORT.

The direct shear at the support, as in any mechanical construction, is equivalent to the total load supported by the column. This shear is readily borne by the concrete and reinforcement. The diagonal tension, however, which, as in a beam, may be considered as measured by direct shear, must be carefully considered. To reduce the diagonal tension and also to increase the resistance to bending of the slab, the column head is enlarged. To still further increase the resistance, a part of the bars in the top of the slab over the supports may be bent down just outside of the supports and then carried along in the bottom of the slab.

In either case, the shearing stress should be limited to definite

units, although it seems permissible to use a somewhat higher stress than in a beam.

The diameter of the enlarged column head, which is the actual support of the slab, should be governed by the shearing stress either at its circumference or at a short distance outside of it.

BENDING MOMENT AT SUPPORT.

The theory of flat plates, which must be used in designing a circular plate, is not yet clearly established. By the use of what

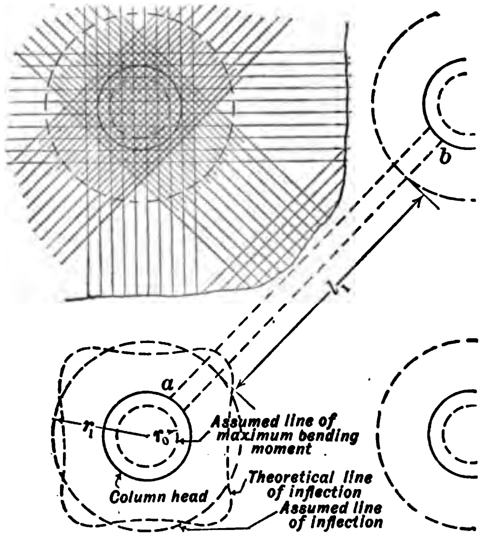


FIG. 1.—PLAN OF FLAT SLAB.

is termed, in mechanics, the elastic theory, we have a fairly good working hypothesis. The analysis solved by Prof. H. T. Eddy* offers, in the writer's judgment, the most rational solution of the problem yet advanced.

In the design of the flat slab, therefore, the author† has started with Prof. Eddy's analysis of stresses in a homogeneous circular plate, and from his general formulas has deduced by mathematics

* Engineers' Society, University of Michigan, 1899.

† The author is indebted to Mr. Edward Smulski for the computations involving intricate analyses by higher mathematics; also to Mr. John Ayer for further studies in the practical design.

other formulas applying to circular plates free on their edges and clamped around the columns. In a flat slab thus supported there are horizontal stresses at right angles to each other. The effect of these lateral stresses has been taken into account, this being expressed by Poisson's ratio, which is the ratio of the lateral deformation to the deformation in the direction of the stress. The value of this ratio is taken as 0.1, which has been shown by experiments to be a fair value for concrete of 1 : 2 : 4 proportions.

It has been found possible to reduce the complicated formulas derived by the Eddy analysis into four formulas which are comparatively simple although still rather complicated for practical use. These formulas are for four bending moments and can be

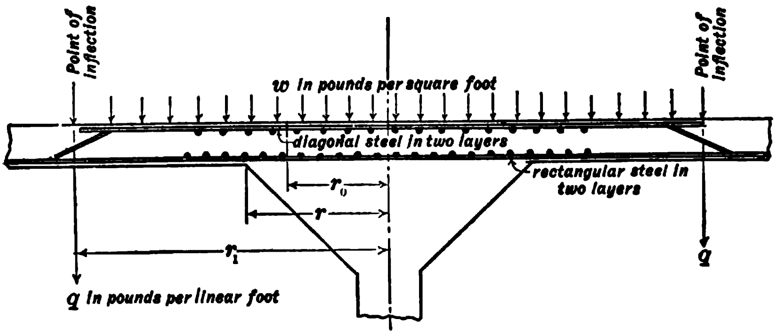


FIG. 2.—SECTION OF FLAT SLAB.

applied not merely to the slab at the support, but to any point in the circular plate surrounding the column. The four moments are as follows:

M_1 = moment produced by the loading that is uniformly distributed over the circular plate and causes circumferential fibre stress.

M_2 = moment produced by this same loading but which causes radial fibre stress.

M_a = moment produced by the loading from the rest of the slab that is distributed along the outer edge of plate and causes circumferential fibre stress.

M_b = moment produced by the latter loading but which causes radial fibre stress.

A study of the analysis, however, shows that the two circumferential moments are a minimum at the support and may be safely disregarded. The two formulas for the radial moment may be combined and still further reduced to the following simple form which can be used for a circle of any radius, r, within the circular plate. The meaning of the symbols is made clearer by reference to Figs. 1 and 2, which show the plan and the section of a flat slab.

Let

q = uniformly distributed load around the outer edge of the plate in lb. per ft. of length.

w = uniformly distributed load on surface of plate (including dead load) in lb. per sq. ft.

r_0 = radius in feet to line of maximum bending moment (which is within the column head).

r_1 = outer radius of assumed plate in feet.

r = any radius in feet where moment is to be computed; for critical section, r is radius of column head.

C_5, C_6 = constants given in Figs. 3 and 4.

M_r = total radial bending moment to be used ordinarily.

l_1 = distance in feet between lines of inflection.

Then total radial moment at any point of plate is

$$M_r = wr_0^2C_5 + qr_0C_6$$

For convenience in computation, values of the constants C_5 and C_6, for various values of the ratios $\frac{r_1}{r_0}$ and $\frac{r}{r}$ are plotted in the curves given in Figs. 3 and 4.*

With q expressed in lb. per ft. of length, w in lb. per sq. ft., and r_0 in ft., the moments are in ft.-lb. per ft. or in.-lb. per in.

POSITION OF MAXIMUM BENDING MOMENT AND OF MAXIMUM STRESS.

As commonly constructed, the column head flares at the top and is therefore more or less flexible. For this reason the line of maximum bending moment will be located, not at the

* These are drawn up from values in tables in Taylor and Thompson's "Concrete, Plain and Reinforced," 2d edition, 1911, page 518.

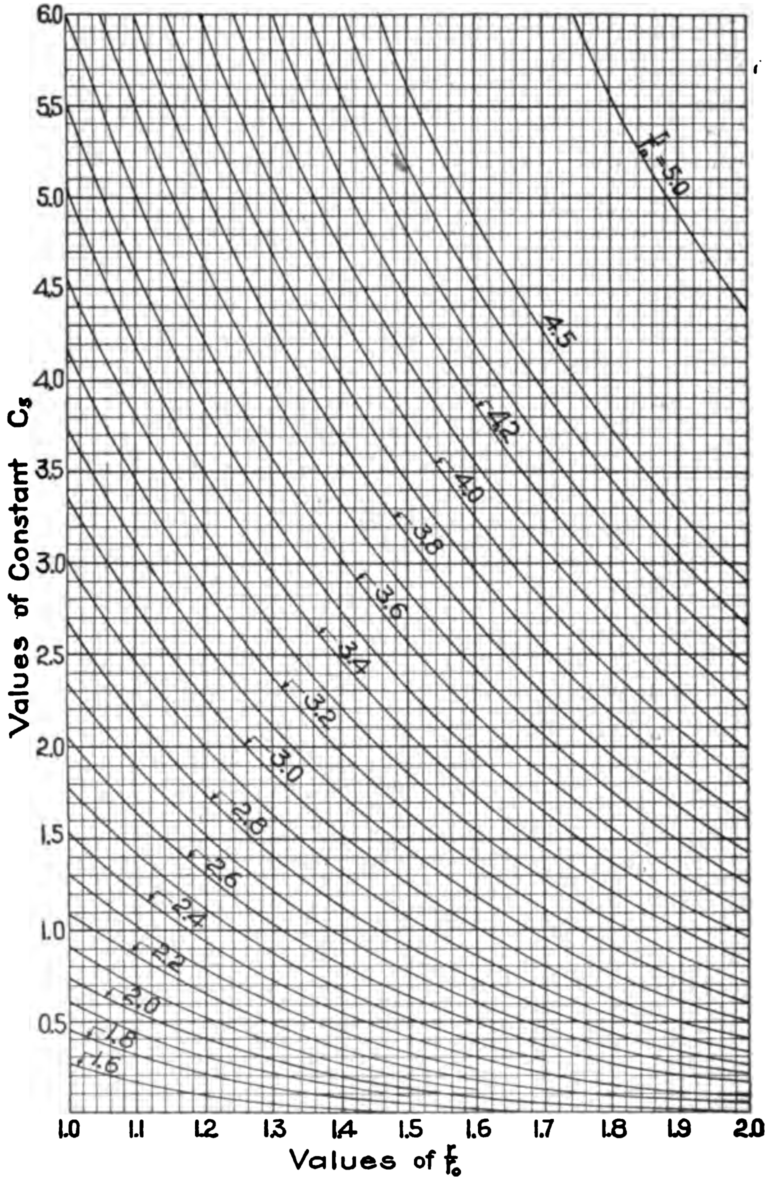


FIG. 3.—DIAGRAM GIVING VALUES OF C_5 IN FORMULA

$$M_r = w\, r_0^2\, C_5 + q\, r_0\, C_6$$

r_0 = radius in feet to line of maximum bending moment.

r_1 = outer radius of assumed plate in feet.

r = any radius in feet where moment is to be computed for actual section ordinarily r is radius of column head.

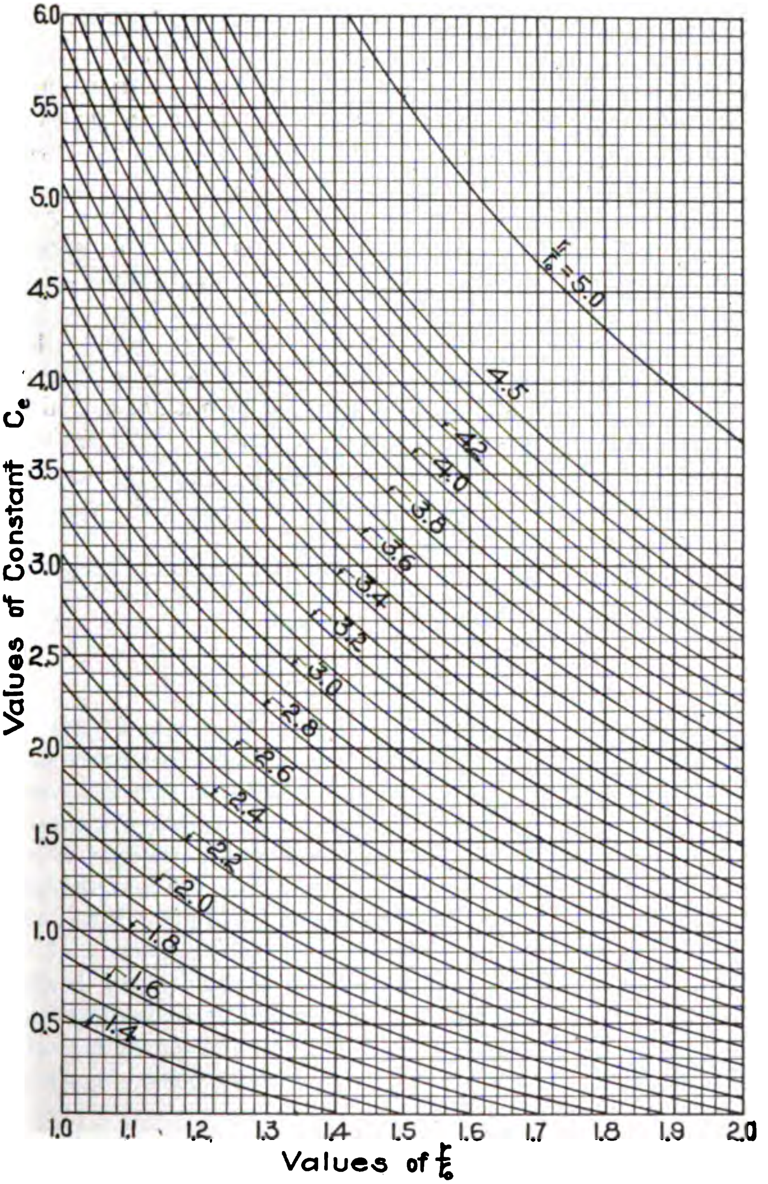


FIG. 4.—DIAGRAM GIVING VALUES OF C_e IN FORMULA

$$M_r = w\, r_0^2\, C_s + q\, r_0\, C_e$$

r_0 = radius in feet to line of maximum bending moment.

r_1 = outer radius of assumed plate in feet.

r = any radius in feet where moment is to be computed for actual section, ordinarily r is radius of column head.

extreme edge of the column head, but a little within it. The maximum stress, on the other hand, will not be on the line of the maximum bending moment because the strength there (since it is within the head) is increased due to the greater depth of concrete. It is fair to assume, therefore, that the maximum stress is at the edge of the column head, and we may assume the "critical section" as on this line. The exact location of the line of maximum moment is indeterminate. Under ordinary conditions it appears fair to assume its location as within the column head, a distance equal to the thickness of the slab. Therefore, M_r is figured for a value of $r=r_0+t$. In figuring this moment, values of the constants C_5 and C_6 should be taken from the curves in Figs. 3 and 4. As in an ordinary fixed beam, this bending moment is negative, so that the upper side of the slab is in tension and the lower in compression. Having found the moment, the design of the reinforcement and the thickness of the slab may be worked out as for an ordinary beam.

The curves in Figs. 5 to 8 inclusive will be found of assistance in working out the design.

REINFORCEMENT IN COLUMN HEAD.*

The slab at the column head might be designed with the reinforcement all in the top of the slab running in four directions provided the slab is thick enough so that the concrete will not be overstressed in compression. In order to reduce the thickness of the slab and therefore save the additional cost and weight of concrete over the entire floor, it is economical to place reinforcement in the bottom of the slab as well as the top, and figure it as assisting the concrete to take compression. Since a portion of the bars need to extend only far enough beyond the column head to furnish suitable bond, the cost of this additional reinforcement will be much less than the cost of an additional thickness of concrete over the entire slab. The tensile reinforcement must not sag over the column head.

To make it easy to place the concrete and also to bring the center of gravity of the reinforcement as near to the surfaces of the slab as possible in order to give the longest moment arm and

* Certain features of flat slab reinforcement are covered by patents of C. A. P. Turner.

thus a thinner slab, two layers of reinforcement may be placed in the top of the slab and two layers in the bottom. The relation of the quantity in the top and bottom must be determined by the design. If a thin slab is desired, even more reinforcement may be placed in the bottom than the top. In the tables, three ratios of reinforcement are given and the percentages selected are those that will give the required working stresses in the concrete and the reinforcement.

The Minneapolis test already referred to shows that not only the reinforcement directly over the column head, but the reinforcement for a considerable distance each side takes tension. In view of this test and of the tests made at the University of Illinois* it is safe to assume that the reinforcement may be spaced over a distance at least equal to the diameter of the column head plus three times the thickness of the slab.

The determination as to whether the diagonal or rectangular reinforcement should be placed at the top is governed by the relative quantities of each. More reinforcement is required for the diagonal direction through the slab, hence the layers which are largest in section may be run diagonally.

AGREEMENT WITH MINNEAPOLIS TESTS.

By our theory it is possible to compute the stresses not only next to the column head but at any point in the slab. In several cases, knowing the exact location of the points where the deformations were measured in the Minneapolis tests the stresses at these points have been computed. Using 5.6 in. as the moment arm, and including the radial bars as assisting to take tension, the maximum stress in the reinforcement over the edge of the column is 25,000 lb. per sq. in. under the normal load of 225 lb. per sq. ft. as compared with 20,700 lb. per sq. in. given by Mr. Lord as the actual maximum stress in the floor. This is no greater difference than there ought to be between design and test and shows our method to be slightly more conservative than the actual test.

The compression in the concrete is more difficult to check since the exact locations of the test points are not given. Com-

* See paper on "A Test of a Flat Slab Floor in a Reinforced Concrete Building," by Arthur, R. Lord, *Proceedings*, Vol. VII, page 182.

putations, however, show unquestionably that our methods are conservative enough to allow for the irregularities in concrete mixtures, and the danger of not having perfect concrete at the critical section.

MOMENT AT CENTER OF SLAB.

It is possible to adapt the Eddy theory to the design of the center of the slab as well as to the supports. In practical design, however, as has been indicated, the thickness of the slab is determined by the thickness at the support, which is always the greater. But, in order to avoid too wide spacing of the bars and to adapt the center reinforcement to that over the supports, more bars are generally run through the slab than the results of tests would show to be necessary. Consequently, instead of considering this from a theoretical standpoint alone, safe values for the bending moments may be selected, based on general principles of mechanics and qualified by actual tests.

Let l_1 = distance between lines of inflection. This distance will be about $\frac{3}{5}$ of the net span between column heads.

For the rectangular reinforcement, if the slabs between the points of inflection were simply supported, we should have a moment of $wl_1^2/8$. However, the bending moment in the Minneapolis tests, based on the maximum stresses under uniform working load, is about $wl_1^2/33$. It would appear amply safe, therefore, to adopt a value of $M = wl_1^2/12$.

For the diagonal reinforcement, the bars run in two directions, and considering both theory and test, a value of $M = wl_1^2/24$ is conservative to use for the reinforcement in each direction.

CROSS REINFORCEMENT BETWEEN COLUMNS.

In flat slab floors cracks are apt to occur between columns on rectangular lines, because, since the span is shorter, the deflection is less than in the center of the slab. To prevent these cracks, it is advisable to place cross reinforcement of small bars in the top of the slab.

TABLES FOR DESIGN OF SLABS.

Tables I–IV give thicknesses of slab, reinforcement and size of column head for various column spacings and loads.

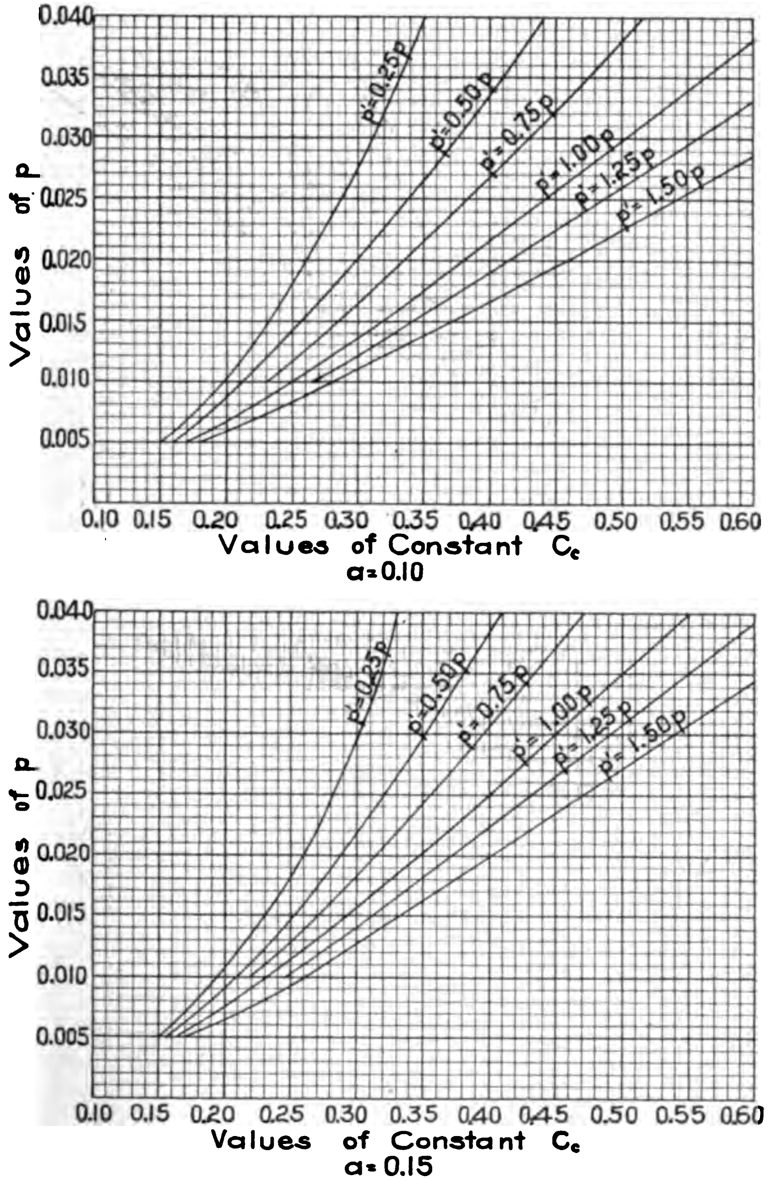
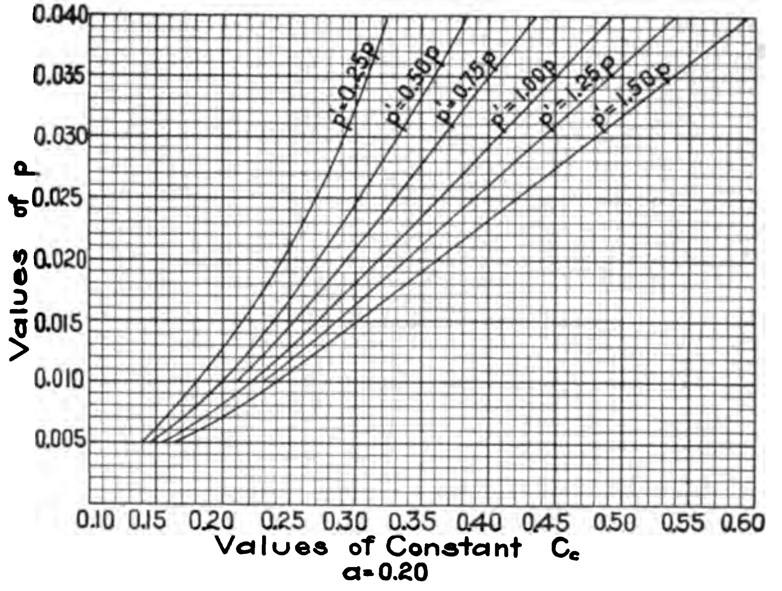


FIG. 5.—DIAGRAM GIVING VALUES OF CONSTANTS IN FORMULA

$$f_c = \frac{M}{C_c\,bd^2} \quad \text{FOR } a = 0.10 \text{ AND } 0.15$$

$$a = \frac{\text{Depth of Reinforcement in Compression.}}{\text{Depth of Reinforcement in Tension.}} \qquad p = \frac{\text{Area of Reinforcement in Tension.}}{\text{Area of Concrete above Reinforcement.}}$$



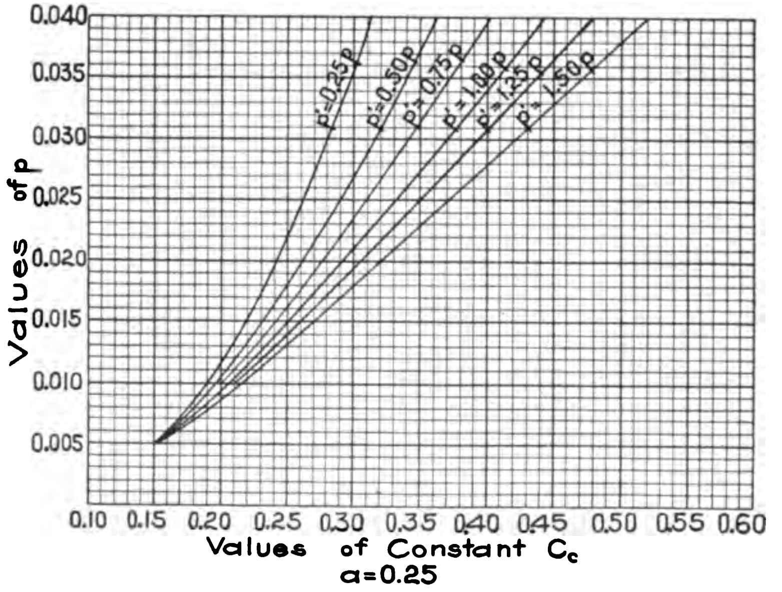


FIG. 6.—DIAGRAM GIVING VALUES OF CONSTANTS IN FORMULA

$$f_c = \frac{M}{C_c\, b d^2} \text{ FOR } a = 0.20 \text{ AND } a = 0.25$$

$$a = \frac{\text{Depth of Reinforcement in Compression.}}{\text{Depth of Reinforcement in Tension.}} \qquad p = \frac{\text{Area of Reinforcement in Tension.}}{\text{Area of Concrete above Reinforcement.}}$$

Three arrangements for reinforcement over the column head are chosen: The first where the area of reinforcement in the top is twice the area of reinforcement in the bottom; the second where the two are equal; and the third where the area of reinforcement in the bottom is one and a half times that in the top. This gives the designer a variety of thicknesses of slab. The

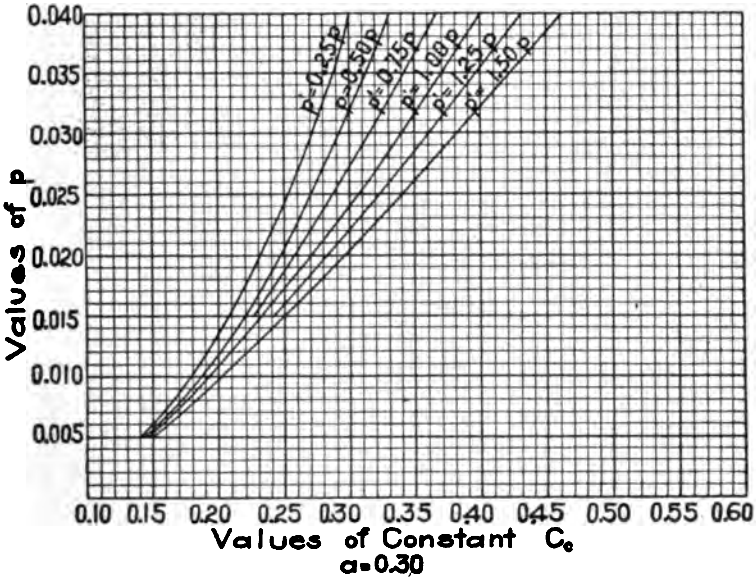


FIG. 7.—DIAGRAM GIVING VALUES OF CONSTANTS IN FORMULA

$$f_c = \frac{M}{C_c\, bd^2} \text{ FOR } a = 0.30$$

$$a = \frac{\text{Depth of Reinforcement in Compression.}}{\text{Depth of Reinforcement in Tension.}}$$

$$p = \frac{\text{Area of Reinforcement in Tension.}}{\text{Area of Concrete above Reinforcement.}}$$

percentages of reinforcement selected are those which produce, with the given conditions, a compressive stress of 800 lb. per sq. in. in the concrete and 16,000 lb. in the reinforcement. In order to allow 800 lb. in the concrete, it should be mixed in proportions as rich as 1 part cement to 2 parts fine aggregate to 4 parts coarse aggregate. Poisson's ratio is assumed as 0.1, which from recent tests appears to be a fair value.

The size of column head has been figured for a shear of 60 lb. per sq. in. on a circle a distance, t, (the thickness of slab) outside of the column head. This shear is used simply as a measure of the diagonal tension. The value is somewhat larger than is permitted in beam design but appears to be warranted in the case of flat slabs.

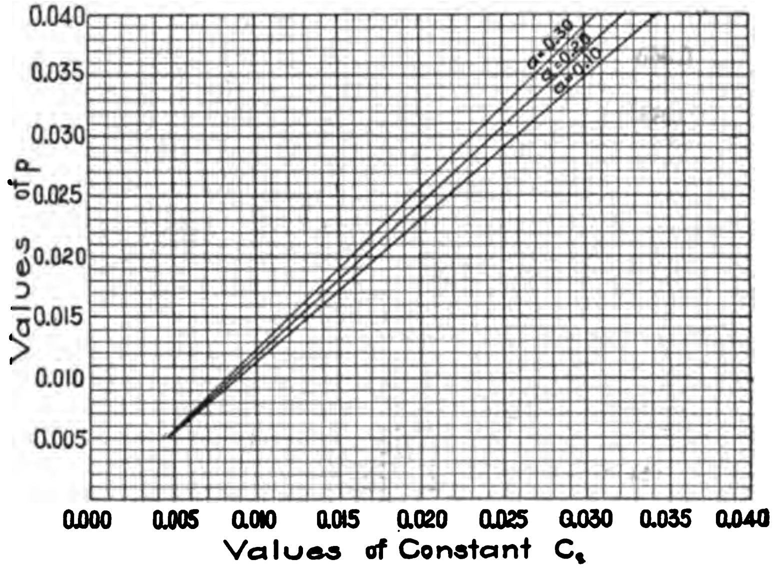


FIG. 8.—DIAGRAM GIVING VALUES OF CONSTANTS IN FORMULA

$$f_s = \frac{M}{C_6\, bd^2}$$

$$a = \frac{\text{Depth of Reinforcement in Compression.}}{\text{Depth of Reinforcement in Tension.}}$$

$$p = \frac{\text{Area of Reinforcement in Tension.}}{\text{Area of Concrete above Reinforcement.}}$$

The reinforcement in the center of the slabs has been figured for a stress of 16,000 lb. per sq. in.

DIAGRAMS FOR DESIGNING SLABS.

To provide for cases not covered by the tables, curves for values of C_5 and C_6 are given so that the moment under various conditions can be readily figured from the formula for the bending moment given above.

DIAGRAMS FOR DETERMINING REINFORCEMENT IN TOP AND BOTTOM OF BEAMS OR SLABS.

In Figs. 5 to 8 curves are plotted for finding the values of the constants C_c and C_s in the formulas for the reinforcement and concrete stresses in beams or slabs with reinforcement in top and bottom. The curves are drawn for different values of a, the ratio of depth of reinforcement in compression to depth of reinforcement in tension, and for different values of p^1/p, where p = ratio of cross-section of reinforcement in tension to concrete above it*, and p^1 = ratio of cross-section of reinforcement in compression to this same area of concrete.

EXAMPLE.

Example.—For a warehouse floor with a live load of 150 lb. per sq. ft. and a column spacing of 20 ft. each way, what is the necessary thickness of slab, size of column head and amount of reinforcement?

Solution.—From Table II the thickness of slab is given as 8½ in., the size of column head as 5.5 ft., and the area of reinforcement as 24.7 sq. in. at top of slab and same amount at bottom of slab over column, using ratio of area of reinforcement in tension to area of concrete below as 0.017. Dividing these values by 4, as each end of the bands is effective, we have 24.7/4 = 6.2 sq. in. as the area of reinforcement in each band. For this may be used twenty ⅝-in. round bars spaced 5 in. center to center for both tension and compression reinforcement.

The amount of reinforcement required at center of rectangular band is 0.17 sq. in. per ft. of width. Placing a ⅝-in. round bar every 10 in. gives more than the necessary area, but ease in placing the reinforcement makes up for the extra amount. The amount required at center of diagonal band is 0.35 sq. in. per ft. of width. ⅝-in. round bars every 10 in. will thus give necessary amount.

* Where the tensile reinforcement is at the top, as over a support of a flat slab or beam, the concrete area is taken below the tensile reinforcement.

TABLE I.—DESIGN OF FLAT SLAB.

THICKNESS OF SLAB, AREAS OF REINFORCEMENT AND SIZES OF COLUMN HEAD ARE GIVEN FOR DIFFERENT SPANS AND PERCENTAGES OF REINFORCEMENT.

LIVE LOAD 100 LB. PER SQ. FT.

Span between center of columns in feet. ft.	Ratio of cross-section of reinforcement in tension to concrete below reinforcement. (p)	Ratio of cross-section of reinforcement in compression to concrete below reinforcement in tension. (p')	Distance from bottom of slab to center of gravity of reinforcement in tension. (d) in.	Approximate total depth of slab. (t) in.	Diameter of column head. ft.	*Area of reinforcement over column in tension. sq. in.	*Area of reinforcement over column in compression. sq. in.	Minimum area of reinforcement between columns per foot of width of diagonal band. sq. in.	Minimum area of reinforcement between columns per foot of width of rectangular band. sq. in.
12	0.014	0.007	4¼	5½	2.00	4.50	2.25	0.16	0.09
12	0.017	0.017	3¾	5	2.00	4.81	4.81	0.17	0.09
12	0.022	0.033	3½	4¾	2.50	7.26	10.90	0.18	0.09
14	0.014	0.007	5	6¼	2.25	5.94	2.97	0.19	0.11
14	0.017	0.017	4½	5¾	2.75	7.93	7.93	0.20	0.11
14	0.022	0.033	4	5¼	3.00	9.96	14.95	0.21	0.11
16	0.014	0.007	6	7½	3.00	9.51	4.76	0.22	0.12
16	0.017	0.017	5¼	6½	3.25	10.95	10.95	0.23	0.12
16	0.022	0.033	4½	5¾	3.75	14.01	21.05	0.24	0.12
18	0.014	0.007	6¾	8¼	3.50	12.48	6.24	0.26	0.14
18	0.017	0.017	6	7½	3.75	14.42	14.42	0.27	0.14
18	0.022	0.033	5	6¼	4.50	18.67	28.00	0.28	0.14
20	0.014	0.007	7¾	9¼	4.00	16.36	8.18	0.30	0.16
20	0.017	0.017	6¾	8¼	4.50	19.47	19.47	0.31	0.16
20	0.022	0.033	5¾	7¼	5.00	23.89	35.80	0.32	0.15
22	0.014	0.007	8½	10¼	4.50	20.21	10.11	0.34	0.18
22	0.017	0.017	7½	9	5.00	24.05	24.05	0.34	0.17
22	0.022	0.033	6½	8	5.75	31.05	46.60	0.35	0.16
24	0.014	0.007	9½	11¼	5.00	25.10	12.55	0.38	0.20
24	0.017	0.017	8¼	10	5.75	30.41	30.41	0.39	0.20
24	0.022	0.033	7	8½	6.50	37.80	56.60	0.40	0.19

* Area of reinforcement over column head = circumference of column head in inches $\times d \times p$ or p' depending upon whether the reinforcement is in tension or compression. This reinforcement is assumed as distributed over the entire widths of the bands. Thus if a band of reinforcement has 2 sq. in. in section the area, effective, for two bands will be 4 sq. in. (See example.)

TABLE II.—DESIGN OF FLAT SLABS.

THICKNESS OF SLAB, AREAS OF REINFORCEMENT AND SIZES OF COLUMN HEAD ARE GIVEN FOR DIFFERENT SPANS AND PERCENTAGES OF REINFORCEMENT.

LIVE LOAD 150 LB. PER SQ. FT.

Span between center of columns in feet. ft.	Ratio of cross-section of reinforcement in tension to concrete below reinforcement. (p)	Ratio of cross-section of reinforcement in compression to concrete below reinforcement in tension. (p')	Distance from bottom of slab to center of gravity of reinforcement in tension. (d) in.	Approximate total depth of slab. (t) in.	Diameter of column head. ft.	*Area of reinforcement over column in tension. sq. in.	*Area of reinforcement over column in compression. sq. in.	Minimum area of reinforcement between columns per foot of width of diagonal band. sq. in.	Minimum area of reinforcement between columns per foot of width of rectangular band. sq. in.
12	0.014	0.007	4¾	6	2.25	5.64	2.82	0.18	0.10
12	0.017	0.017	4¼	5½	2.50	6.81	6.81	0.19	0.10
12	0.022	0.033	3¾	4¾	3.00	8.72	13.09	0.21	0.10
14	0.014	0.007	5½	7	3.00	8.72	4.36	0.22	0.12
14	0.017	0.017	5	6¼	3.50	11.22	11.22	0.23	0.11
14	0.022	0.033	4¼	5½	3.75	13.22	19.82	0.24	0.11
16	0.014	0.007	6¼	8	3.50	12.03	6.02	0.26	0.14
16	0.017	0.017	5¾	7¼	3.75	13.83	13.83	0.27	0.13
16	0.022	0.033	4¾	6	4.50	17.75	26.80	0.28	0.13
18	0.014	0.007	7¼	8¾	4.00	15.32	7.66	0.31	0.16
18	0.017	0.017	6¼	7¾	4.50	18.05	18.05	0.32	0.15
18	0.022	0.033	5¼	6¾	5.50	24.00	36.00	0.33	0.17
20	0.014	0.007	8¼	10	5.00	21.80	10.90	0.34	0.17
20	0.017	0.017	7	8½	5.50	24.70	24.70	0.35	0.17
20	0.022	0.033	6	7½	6.25	31.14	46.70	0.36	0.16
22	0.014	0.007	9	10¾	5.50	26.15	13.08	0.38	0.18
22	0.017	0.017	7¾	9¼	6.25	31.07	31.07	0.39	0.17
22	0.022	0.033	6¾	8¼	7.00	39.24	58.80	0.40	0.17
24	0.014	0.007	9¾	11½	7.00	36.05	18.03	0.42	0.21
24†	**0.016**	**0.013**	**8¾**	**10½†**	**7.00**	**36.95**	**29.57**	**0.43**	**0.20**

The values printed in black type are figured for a column head 7 ft. in diameter and the thickness of the slab is increased to withstand the shear.

* Area of reinforcement over column head = circumference of column head in inches $\times d \times p$ or p^1 depending upon whether the reinforcement is in tension or compression. This reinforcement is assumed as distributed over the entire widths of the bands. Thus if a band of reinforcement has 2 sq. in. in section the area, effective, for two bands will be 4 sq. in. (See example.)

† The thickness of slab for the 24-ft. span may be decreased to 8¾ in. by using 0.022 and 0.033 ratios of reinforcement and bending the bars to resist diagonal tension.

TABLE III.—DESIGN OF FLAT SLABS.

THICKNESS OF SLAB, AREAS OF REINFORCEMENT AND SIZES OF COLUMN HEAD ARE GIVEN FOR DIFFERENT SPANS AND PERCENTAGES OF REINFORCEMENT.

LIVE LOAD 200 LB. PER SQ. FT.

Span between center of columns in feet. ft.	Ratio of cross-section of reinforcement in tension to concrete below reinforcement. (p)	Ratio of cross-section of reinforcement in compression to concrete below reinforcement in tension. (p')	Distance from bottom of slab to center of gravity of reinforcement in tension. (d) in.	Approximate total depth of slab. (t) in.	Diameter of column head. ft.	*Area of reinforcement over column in tension. sq. in.	*Area of reinforcement over column in compression. sq. in.	Minimum area of reinforcement between columns per foot of width of diagonal band. sq. in.	Minimum area of reinforcement between columns per foot of width of rectangular band. sq. in.
12	0.014	0.007	5	6¼	2.50	6.60	3.30	0.20	0.10
12	0.017	0.017	4½	5¾	3.25	9.38	9.38	0.21	0.10
12	0.022	0.033	3¾	5	3.75	10.37	15.40	0.23	0.10
14	0.014	0.007	6	7½	3.25	10.31	5.16	0.24	0.13
14	0.017	0.017	5	6¼	3.75	12.02	12.02	0.25	0.12
14	0.022	0.033	4¼	5½	4.50	15.88	23.80	0.26	0.11
16	0.014	0.007	6¾	8¼	4.00	14.26	7.30	0.28	0.14
16	0.017	0.017	5¾	7¼	4.50	16.60	16.60	0.30	0.14
16	0.022	0.033	4¾	6	5.50	21.70	32.55	0.32	0.14
18	0.014	0.007	7½	9	4.75	18.80	9.40	0.33	0.17
18	0.017	0.017	6½	8	5.50	22.92	22.92	0.34	0.16
18	0.022	0.033	5½	7	6.50	29.70	44.60	0.35	0.15
20	0.014	0.007	8½	10¼	5.50	24.71	12.36	0.37	0.18
20	0.017	0.017	7¼	8¾	6.25	29.08	29.08	0.38	0.17
20	**0.019**	**0.029**	**6¼**	**7¾**	**7.00**	**31.35**	**47.85**	**0.39**	**0.16**
22	0.014	0.007	9½	11¼	6.25	31.38	15.69	0.42	0.20
22†	**0.016**	**0.012**	**8¼**	**10†**	**7.00**	**34.90**	**26.15**	**0.42**	**0.19**
24†	**0.014**	**0.006**	**10¼**	**12†**	**7.00**	**37.90**	**16.24**	**0.46**	**0.22**

The values printed in black type are figured for a column head 7 ft. in diameter and the thickness of the slab increased to withstand the shear.

* Area of reinforcement over column head = circumference of column head in inches $\times d \times p$ or p' depending upon whether the reinforcement is in tension or compression. This reinforcement is assumed as distributed over the entire widths of the bands. Thus if a band of reinforcement has 2 sq. in. in section the area, effective, for two bands will be 4 sq. in. (See example.)

† The thickness of slabs for the 22- and 24-ft. spans may be decreased to 8¼ in. and 9½ in. respectively, by using 0.022 and 0.033 ratios of reinforcement and bending the bars to resist diagonal tension.

TABLE IV.—DESIGN OF FLAT SLABS.

THICKNESS OF SLAB, AREAS OF REINFORCEMENT AND SIZES OF COLUMN HEAD ARE GIVEN FOR DIFFERENT SPANS AND PERCENTAGES OF REINFORCEMENT.

LIVE LOAD 300 LB. PER SQ. FT.

Span between center of columns in feet.	Ratio of cross-section of reinforcement in tension to concrete below reinforcement.	Ratio of cross-section of reinforcement in compression to concrete below reinforcement in tension.	Distance from bottom of slab to center of gravity of reinforcement in tension.	Approximate total depth of slab.	Diameter of column head.	*Area of reinforcement over column in tension.	*Area of reinforcement over column in compression.	Minimum area of reinforcement between columns per foot of width of diagonal band.	Minimum area of reinforcement between columns per foot of width of rectangular band.
	(p)	(p')	(d)	(t)					
ft.			in.	in.	ft.	sq. in.	sq. in.	sq. in.	sq. in.
12	0.014	0.007	5¼	6½	3.50	9.72	4.86	0.24	0.12
12	0.017	0.017	4½	5¾	4.25	12.27	12.27	0.25	0.11
12	0.022	0.033	3¾	5	5.00	15.56	23.30	0.26	0.10
14	0.014	0.007	6¼	7¾	4.25	14.03	7.02	0.29	0.13
14	0.017	0.017	5¼	6½	5.00	16.84	16.84	0.30	0.12
14	0.022	0.033	4¼	5½	6.00	21.18	31.72	0.31	0.11
16	0.014	0.007	7	8½	5.00	18.48	9.24	0.33	0.15
16	0.017	0.017	6	7½	6.00	23.10	23.10	0.35	0.15
16	0.022	0.033	5	6¼	7.00	29.05	43.50	0.35	0.12
18	0.014	0.007	7¾	9½	6.00	24.59	12.30	0.39	0.17
18	0.017	0.017	6½	8	7.00	29.18	29.18	0.40	0.16
18	**0.015**	**0.015**	**6¾**	**8¼**	**7.00**	**26.74**	**26.74**	**0.39**	**0.15**
20	0.014	0.007	8¾	10½	7.00	32.32	16.16	0.43	0.19
20†	**0.015**	**0.011**	**8½**	**10¼†**	**7.00**	**33.65**	**25.36**	**0.43**	**0.18**
22†	**0.012**	**0.003**	**10¾**	**12½†**	**7.00**	**32.61**	**8.23**	**0.47**	**0.22**
24†	**0.010**	**0.000**	**13½**	**15½†**	**7.00**	**35.64**	**0.00**	**0.51**	**0.25**

The values printed in black type are figured for a column head 7 ft. in diameter and the thickness of the slab increased to withstand the shear.

* Area of reinforcement over column head = circumference of column head in inches $\times d \times p$ or p' depending upon whether the reinforcement is in tension or compression. This reinforcement is assumed as distributed over the entire widths of the bands. Thus if a band of reinforcement has 2 sq. in. in section the area, effective, for two bands will be 4 sq. in. (See example.)

† The thickness of slabs for the 20-, 22- and 24-ft. spans may be decreased to 8¼ in., 9¼ in. and 11 in. respectively, by using 0.022 and 0.033 ratios of reinforcement and bending the bars to resist diagonal tension.

DISCUSSION

Mr. Lindau.

Mr. Alfred E. Lindau.—Regarding this method of designing a flat slab or plate of concrete it might be of value to note that the analysis is based on the assumption that there exists a line of contra-flexure around the column substantially circular in form and near the quarter point of span, that the balance of the span is supported along this line and that consequently there is no stress in any direction along this line. There is some question as to this line of contra-flexure, in fact there is some evidence to show that instead of a line of contra-flexure there exist perhaps only four points where there is no stress in any direction. If such is the case the analysis would be entirely different from that which has been outlined.

Mr. Anderson.

Mr. W. P. Anderson.—One of the important things in designing a flat slab has not been touched—the bending moment on the exterior column. I think that has an effect on the thickness of the slab. The thicker the slab, it appears to me, the less that bending moment would be. I have not gone into the matter thoroughly enough to determine how the bending moment of the exterior column is to be figured, but it seems to me that it is the critical point in the design of the flat slab and one that has not yet been touched upon enough by those who have given the subject thought. This point in the design of the flat slab has not been covered in the tests made on the flat slabs. They have been made in the center of the building and the exterior column has not been tested at all to see what the moment is. It is something that ought to be covered. I am not prepared to say how it ought to be taken care of, but I believe it ought to be looked into.

Mr. Lindau.

Mr. Lindau.—The test covered in Part II of the report of the Committee on Reinforced Concrete refers to an end or wall panel. In fact, it was a building with one line of columns making all panels end panels, so to speak.

Mr. Green.

Mr. Herbert P. Green.—I have been experimenting a little bit this last year on flat slabs of a different construction

from any others I have seen. Realizing that the shear around the head of the column is the greatest thing to be considered, I endeavored to work out a way of thickening the slab at this point and have succeeded in obtaining a flat ceiling without the flaring head. We have built one building by this system and tested it and it is a perfect success. Mr. Green.

The reinforcing bars run directly and diagonally from column to column. The bending moment on each beam is considered as coming from the load of one-fourth of the panel, which positively occurs because triangles are formed in between the direct and diagonal girders which are filled with ordinary stock tile. Between these tile are concrete joists, supported by the beams. This construction can be figured by any engineer without empirical formulas.

At the center of the slab between the columns on the diagonal girders in the bottom or top of the slab, supplementary reinforcement can be placed, but it is generally not necessary. Every beam and every girder, or every joist and every beam, as they may be called, is of a T-section, the concrete above the tile forming the flange of the direct and diagonal beams and the supplementary joists in the triangles, so that the analyses of the stresses in this slab are very easy, according to the ordinary theories of design.

MR. LINDAU.—I would like to ask Mr. Green whether in his method of flat slab construction he has made joints along these triangles, so that the tile filler is disconnected absolutely from the balance of the slab thereby making separate beams, or whether it is all concreted in and becomes a portion of the slab as a whole. Mr. Lindau.

MR. GREEN.—The tile actually becomes a portion of the slab, but—according to the methods generally used for figuring the efficiency of tile joist construction—the tile is considered as nothing but a core, so that the concrete construction is the principal thing considered. If we should remove the tile, in this construction it would be the same thing as removing it in any tile joist construction, *i. e.*, we would have a floor with ribs or stiffeners below, and in this case they would be in four directions from column to column, with supplementary stiffeners in the triangles. The tile is simply a form. Mr. Green.

Mr. Lindau.

MR. LINDAU.—I take exception to the assumption that you have a series of beams that are free and independent to act as beams; because just as soon as you connect up the slab so as to have anything resembling a homogeneous structure you have changed the stress condition from a slab to a beam. The loads are transmitted by bending moment or transverse stresses, or a combination of those two; you cannot draw imaginary diagonals or anything on that slab and say that this is the direction in which the load will travel. The load will travel along the lines of relative rigidity of the structure and you cannot make it go in one direction rather than another unless the slab is built accordingly. This is the difficulty with most theories and methods of strip analysis, where the freedom of the slab is tied up in various ways and consequently cannot be separated into the various elements it may be considered to be made up of.

Mr. Green.

MR. GREEN.—That may be so, but in the two-way reinforced concrete floor with tile fillers one figures the loads are carried through the joists, which are supported on the beams, to the beams which run directly between the columns. Now I simply make two more beams or headers in the slab. I have run them diagonally across from column to column and the joists in the triangles between the direct and diagonal beams are supported by these direct and diagonal beams. It is the same principle that would occur in a wooden framed structure with diagonal beams, except that we have a T-section in all directions and the stresses in the concrete above and below the neutral axis at any point can be figured just as readily as in the two-way reinforced concrete floor.

THE DESIGN OF CONCRETE GRAIN ELEVATORS.

By E. Lee Heidenreich.*

During the International Congress of Engineers in 1893, at Chicago, Ill., I was appointed by the American Society of Civil Engineers to present a paper on "American Grain Elevators," having at that time had about ten years' experience in the construction and erection of grain elevators in North and South America. Elevators at that time had the same functions as they have today and may as a rule be divided into certain classes:

1. Farmers' elevators or small station elevators with a capacity from 5000 to 50,000 bushels—where the farmers would deliver their grain in wagon loads, up to 5 tons capacity, have it weighed, elevated and shipped out in railroad cars.

2. Mill elevators or cleaning elevators, built as adjuncts to flour mills, within varying capacities from 50,000 to 500,000 bushels and containing, besides receiving and shipping apparatus, scales and cleaning machinery.

3. Terminal elevators, divided into storage and working houses. The working houses would receive the grain, weigh and ship it into the storage elevators or cars. The storage elevators would merely be for the purpose of storing grain.

4. Transfer elevators located at prominent points where different grains are graded and changed from one grade to another and shipped from western to eastern cars.

5. Marine elevators, where the grain would either be received from canal boats or vessels or shipped into them. These elevators are either dock elevators or floating elevators.

Up to 1893 there were no reinforced concrete grain elevators in the United States. At that time two large grain elevators were under construction at Galatz and Braila, Roumania, and Mr. Herman O. Schlawe, a representative of the Roumanian government, sent over to see how we handled grain and took care of our grain elevators, showed me the plans of the same elevators designed by Luther of Braunschweig, Germany. These elevators

*Consulting Engineer, Kansas City, Missouri.

were of unit construction, somewhat similar to the one described by Mr. Darnell,* excepting that the bins were honeycombed hexagonal cells, built of flat slabs 3 ft. square, and corner pieces, all interlocked. They were all built on the ground, erected and interlocked somewhat similar to tile construction. The difficulty with flat slab walls was, of course, that they were subject to direct flexion, and therefore became quite cumbersome, owing to the lateral, rather heavy pressure in grain bins, and were only suitable for bins of small dimensions.

Steel tanks have been tried in this country, but there was more or less sweating of the tank, which to some extent damaged the grain. Circular reinforced concrete tanks were then designed, using the space between the tanks whereby considerable economy of ground area was effected as compared with isolated tanks and thereby increasing their combined strength and carrying capacity on the soil or the sub-structure. The introduction of the new design was, however, exceedingly difficult.

The owners thought the tanks would sweat and damage the grain and builders thought the tanks would surely burst. From 1896 to 1899 I designed some scores of cluster tank elevators and presented them to millers and grain elevator owners entirely in vain, until in 1899, Frank H. Peavey, of Minneapolis, as an experiment built a tank 23 ft. in diameter 130 ft. high, filled it with grain and found that it held grain without spoiling. He then sent some representatives over to Europe to see what effect grain storage in reinforced concrete had in Europe; they examined the tanks and grain elevators at Galatz and Braila, in Roumania, and came back with a report which resulted in the construction, in 1901, of a large one million and a half bushel elevator in Duluth. In the meanwhile I had built, in 1900, a cement storage elevator at South Chicago, consisting of four tanks 25 ft. in diameter and 56 ft. high, clustered in such manner as to utilize the space between them.

It must be remembered that wheat weighs 50 lb. per cu. ft. while cement weighs 100 lb.; and the fact that a cement tank with 5-in. walls at the top and 7-in. at the bottom, 25 ft. in diameter, would hold cement, went a long step towards convincing people that it was safe to store grain in circular tanks and also in cluster

* See p. 464.—ED.

tanks. The tanks in South Chicago were erected on top of reinforced concrete girders, which in turn rested on columns, thus forming a working house underneath where the bagging of the cement took place.

Since that time cluster tanks have grown up like mushrooms all over this country, Canada, South America and Europe. There are upwards of 60,000,000 bushels capacity built in the United States alone to-day and they are being built in every part of the country for terminal elevators and largely for milling elevators, giving safe and good storage and eliminating the question of insurance and the calamity due to fire or burning up of the storage plant.

The design of reinforced concrete grain elevators, consisting of circular tanks, has been carefully studied. It is of course entirely a function of the grain pressure in rest and in motion. Experiments and calculations have been made by a number of well-known engineers. Janssen, Wilfred Airy, Tolz, Prante, Jameson of Canada and Milo B. Ketchum, have all made experiments and developed formulas whereby the grain pressure may be determined at the different heights of the bin. Grain is not like a liquid—Professor Ketchum calls it a semi-liquid,—the pressure line forms a curve.

Inasmuch as Janssens' solution tallies very closely with Jameson's experiments, we may write the lateral pressure

$$L = \frac{wR}{\mu}\left(1 - e^{\frac{-C\mu' h}{R}}\right)$$

and the vertical pressure

$$V = \frac{wR}{C\mu'}\left(1 - e^{\frac{-C\mu' h}{R}}\right)$$

Where w = the weight per cu. ft. of grain or 50 lb. for wheat,
R = bin area divided by its perimeter (hydraulic radius).
μ' = the coefficient of friction of grain against bin surface;
h = the height of the bin, and
C = the ratio between the lateral and the vertical pressure;
e = being the base of the Naperian logarithm or 2.71828.

According to Jameson for wheat $C = 0.6$ and for grain on concrete $\mu' = 0.4$ to 0.425.

For quick and easy calculation the lateral pressure per sq. ft. $= q\,w\,h$, where q is the ratio of grain pressure to liquid pressure and

$$V = \frac{q\,wh}{C} = 1.667\,q\,wh$$

Fig. 1 shows the value of q for different ratios of height

WHEAT PRESSURES IN GRAIN BINS.

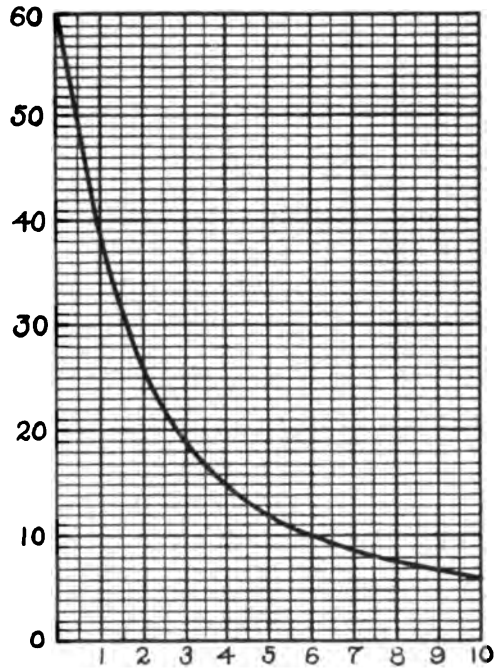


FIG. 1.—VALUES OF Q FOR DIFFERENT RATIOS OF HEIGHT TO DIAMETER OF BIN.

divided by the diameter or width $= \frac{h}{b}$. The maximum bottom pressure occurs when $\frac{h}{b} = 3.5$.

Fig. 2 shows a series of circular bins with intervening spaces. The lateral pressure per sq. ft. of the grain in a circular bin at a depth h is equal to

$$L = q\,wh$$

and the tension in the reinforcement required for 1 ft. in height is

$$T = \frac{L\,D}{2}$$

The reinforcement area in sq. ins.

$$A_s = \frac{T}{f_s}$$

where f_s is the working stress of the reinforcement per sq. in.

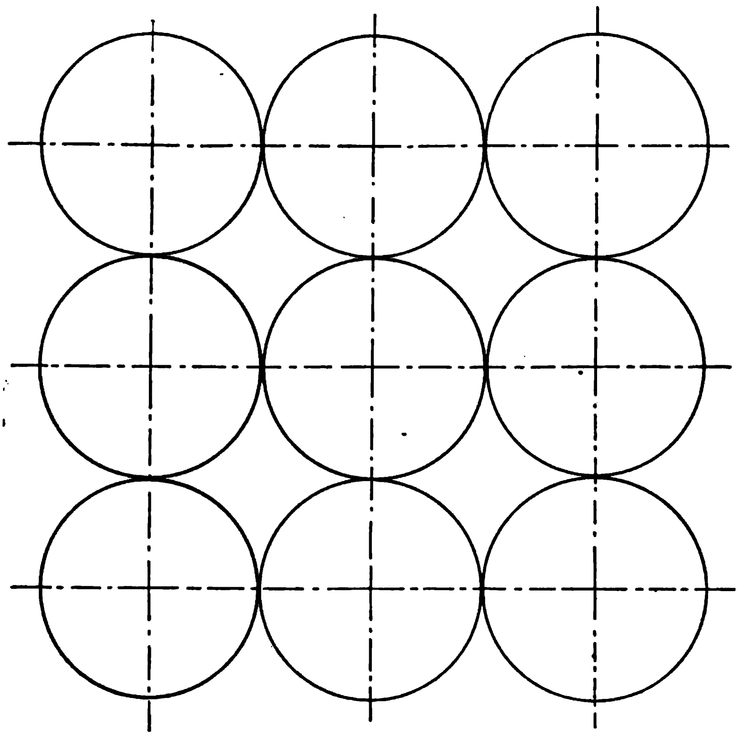


FIG. 2.—SERIES OF CIRCULAR BINS.

If, however, the circular bin is empty and the two opposite interstices A and B filled, we have a condition as shown in Fig. 3 where it is quite apparent that instead of single reinforcement of the bin, there should be reinforcement both at the intrado and extrado of the ring.

Compression in the direction A B clearly causes tension at

the extrado at C and D, and at the intrado A and B. From experiments in loading culvert pipe firmly supported at the two lower quarter points corresponding to E and F, the maximum moment may approximately be written (Fig. 4):

$$M = \frac{\left(\frac{D}{\sqrt{2}} - 6\right) L'D'}{64}$$

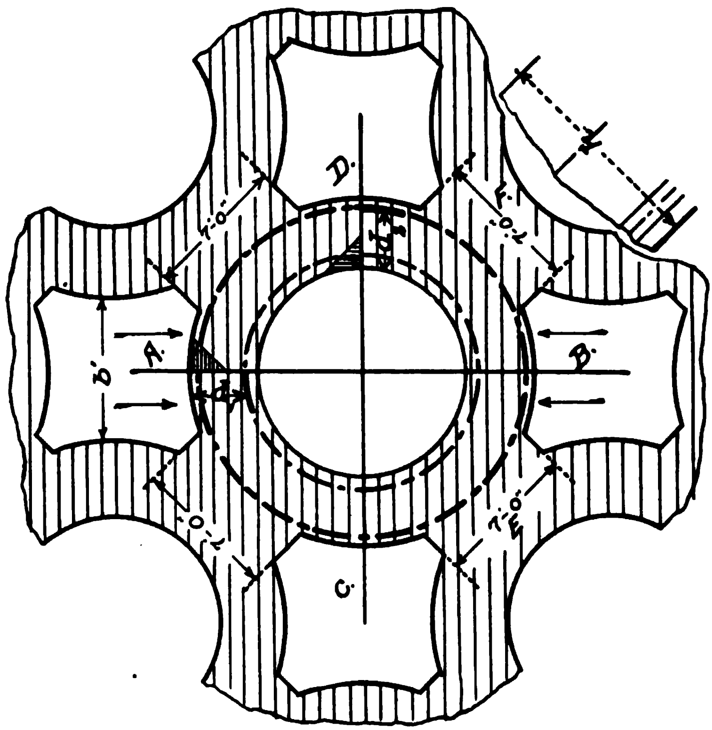


FIG. 3.—PLAN OF CIRCULAR BIN.

The usual formula for the resisting moment is

$$M = Kbd^2 \text{ and } K = \frac{1}{2} f_c k \left(1 - \frac{k}{3}\right) = pf_s \left(1 - \frac{k}{3}\right)$$

where

$$k = \sqrt{n^2p^2 + 2\,np} - np,\ n = \frac{E_s}{E_c} = 15,\ p = \frac{A_s}{bd},\ A_s = \frac{M}{jf_s\,d},\ j = \left(1 - \frac{k}{3}\right) = 0.86$$

f_s and f_c the unit stresses in reinforcement and in concrete.

The stress in the concrete or in the reinforcement may be expressed by

$$f_c=\frac{M}{\frac{1}{2}k\left(1-\frac{k}{3}\right)bd^2} \quad \text{and} \quad f_s=\frac{M}{p\left(1-\frac{k}{3}\right)bd^2}$$

By adding 1 in. to d_1 or d_2, the ring thickness is found.

Example: The 25 ft. diameter bins are 80 ft. deep and surrounded by interstices, find the dimensions of concrete wall and reinforcement required at 60 ft. from the top.

$$\frac{h}{b}=\frac{60}{25}=2.4$$

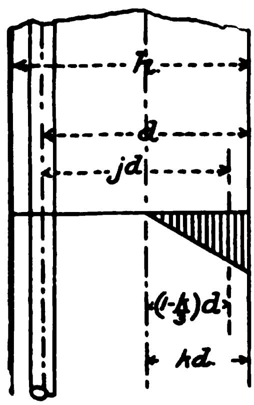


FIG. 4.

hence

$$q=0.23 \text{ and } L=0.23\times50\times60=690 \text{ lb. per sq. ft.}$$

Then

$$T=\frac{690\times25}{2}=8625 \text{ lb.}$$

and

$$A_s=\frac{8625}{16000}=.054 \text{ sq. in. per each foot high}$$

The inner and outer reinforcement from the effects of grain in the center bin is

$$\frac{0.54}{2}=0.27 \text{ sq. in. or say } \tfrac{1}{2}\text{-in. rounds } 8\tfrac{1}{2} \text{ in. on centers}$$

Vertical rods ½-in. rounds 24 in. on centers tied to horizontals.

For external pressure we have

$$\frac{h}{b'}=\frac{60}{10}=6 \text{ hence } q=0.1 \text{ and } L=0.1\times 50\times 60=300 \text{ lb. per sq. ft.}$$

$$M=\frac{\left(\frac{D'}{\sqrt{2}}-6\right)L'D'}{64}=\frac{\left(\frac{26}{1.41}-6\right)300\times 26\times 12}{64}=18700 \text{ in. lbs.}$$

$$d_1=d_2=\sqrt{\frac{18700}{k\times 12}}=\sqrt{\frac{18700}{110\times 12}}=3.85 \text{ in.}$$

hence the wall should be 5 in. thick.

$$A_s=\frac{18700}{0.86\times 16000\times 4}=0.34 \text{ sq. in. for } each\text{, both extrado and intrado,}$$

showing that the stresses from the filled interstices are greater than those resulting from the filled circular bins. Some contractors using single reinforcement in the center bins have rodded the interstices, others increase the thickness of the bin walls and the connection between contiguous circular bins.

The logical method, would, however, seem to be to use double reinforcement, a practice the author invariably prefers for circular culverts.

As to the calculation of square bins, this is a simple matter, after the lateral pressure L has been found.

The bending moment will be in *in.-lbs.*

$$M=\frac{Ll^2}{10}12, \text{ where } l=\text{width of bin in } feet.$$

The resisting moment as before

$$M=kbd^2 \text{ and } d=\sqrt{\frac{M}{kb}}$$

Area of reinforcement required

$$A_s=\frac{M}{jf_sd} \text{ as before}$$

Interior bin walls having alternately pressure on either side are, of course, reinforced on both sides. The connection of the reinforcement at the intersections and the corners of the bin walls become of the greatest importance. Where possible the author prefers to employ a strong wire fabric as a part of the reinforce-

ment for the outer walls, running this reinforcement horizontally around the entire building.

Fig. 5 shows a common country elevator of 15,000 bushels capacity. The structure rests upon a slab or raft extending under the entire building and connected with the side walls so as to make the pit absolutely waterproof. The concrete is mixed

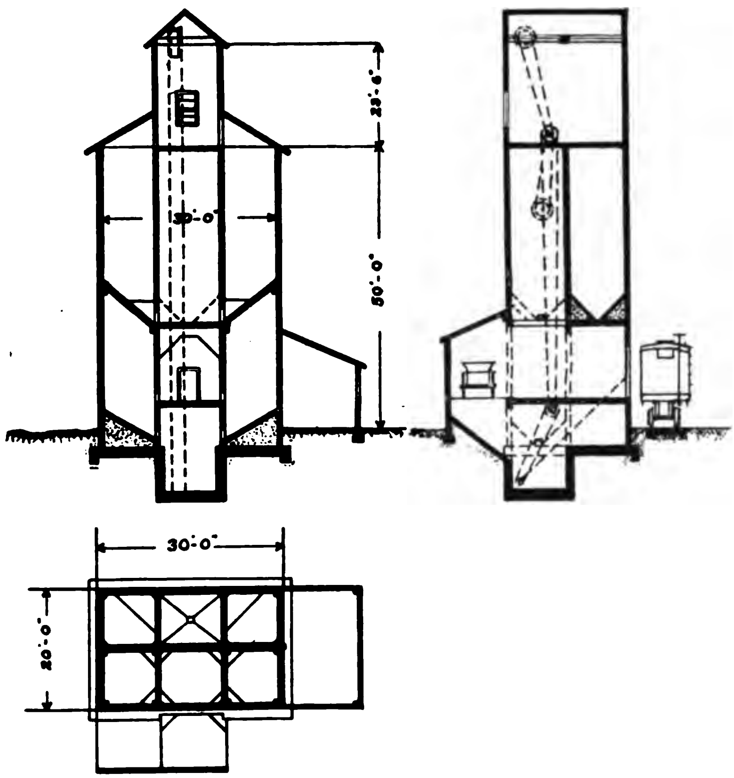


FIG. 5.—CONCRETE COUNTRY ELEVATOR.

1 : 2 : 4 to maximum density and to each bag of cement is added 5 lb. of petroleum residuum oil which has been found an excellent and cheap method of making the walls and roof impervious to moisture. The roof and cupola walls are reinforced with fabric to prevent cracks by shrinkage.

Fig. 6 shows an outside wall connection and the reinforcement, illustrating how the horizontal and hooked members are tied to the vertical rods, to insure their proper position until

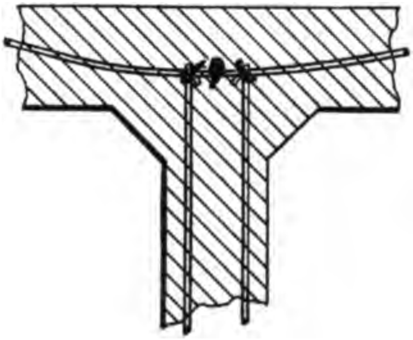


FIG. 6.—OUTSIDE WALL CONNECTION OF REINFORCEMENT.

the concrete has been poured. The hooks of the corner rods (Fig. 7) are placed by springing the vertical rods until the corner rods can be hooked in and thereby form a support for the outside carrying rods.

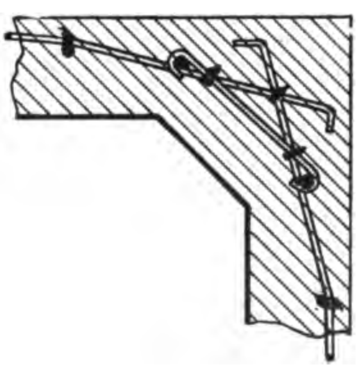


FIG. 7.—METHOD OF HOOKING CORNER REINFORCEMENT.

In 1911 the corn crop of the United States was..	2,531,488,000	bu.
and of wheat..............................	631,388,000	"
Total of..............................	3,152,786,000	"

Most of this crop was handled through farmers' elevators, averaging, say 150 cars a year, or 150,000 bu., meaning a total of approximately 20,000 farmers' grain elevators in the country. Most of these are built in wood and will ultimately have to be replaced with concrete construction, which in turn shows one immediate channel for the use of Portland cement. The cost of a 150,000-bushel grain elevator in concrete is about $5;000.00 exclusive of machinery and millwrighting. Considering the thousands of wooden elevators rotting away and burning up along our western trunk and grain lines—the almost irreparable loss to a town or community of farmers when its elevator burns down, the annual cost of insurance and maintenance of these wooden elevators—the great doctrine of conservation brings into the limelight the impending want of reinforced concrete grain elevators in every nook and corner of the land.

DISCUSSION.

Mr. Lindau.

MR. ALFRED LINDAU.—In the design of circular bins for exterior pressure of the space between the bins, is account taken of the support that is afforded by the adjacent bins? As I understand, the worst condition is where the two opposite spaces would be filled with grain and others would not be filled with grain; but the bin, I understand, is supported on the other diameter by the adjacent bins. Is this taken into consideration in the strength of the structure, or merely left as an added factor of safety. What are the general principles that govern the economy of grain storage? Bins are built high and you can build them wide, circular or square. Has an investigation been made to show the design or adopt a bin that would attain economy for storage purposes?

Mr. Heidenreich.

MR. E. LEE HEIDENREICH.—The location of the adjacent bins is taken into consideration, otherwise we could use Professor Talbot's formula divided by 16, $wd/16$, in place of dividing by 64—an empirical factor that I brought in.

There are many items, many functions, which come into play in the determination of a grain elevator. First, the storage capacity required. It is possible to store grain in very large bulk, requiring very large bins. I have built them up to 30,000 and 40,000 bushel bins for winter storage. In other cases there are what we call pocket bins, carload bins, which are required where certain shippers want their grain individually, a practice quite common. Then again comes the floor space. In a city the ground space is very expensive and it is important to economize, and elevators are built as high as possible, as high as the soil will carry them. If the soil will not carry them they are put on timber piling under all piers. In other places, again, you can spread the bins considerably, but, as a general rule, to utilize the gravity of the grain in handling it throughout the elevator is considered the best economy. For instance, in a cleaning elevator you build it so high that from the top of the elevator head the grain runs first into the garner, then into the scales, then into the

cleaning bin; from there through the cleaners and separators to receiving bins and then into the loading apparatus. But you gain by it, because the grain goes by the gravity from the top in one process to the bottom in the place of being re-elevated, which consumes power and of course adds to the cost of handling of the grain. Mr. Heidenreich.

There are a great many other elements entering into shape of grain elevators, but these are a few of the functions that must be considered.

MR. WM. M. KINNEY.—In bins holding cement, there is often a sweeping action, similar to a wave action. Mr. Heidenreich spoke of the semi-fluid condition of grain, or an action like a semi-fluid. In a cement bin the cement will hang up on one side and all at once give way and sweep clear to the other side, across to the opposite side wall of the tank, and put a strain there that would have to be figured on. I was wondering whether there was anything like that in the bins. Mr. Kinney

MR. HEIDENREICH.—This only happens with wet oats; they act very much like cement in bulk. On account of that action instead of building the hoppers converging to the center in a cement storage elevator at South Chicago, they were built diverging to the sides, like over a peak in the center of the bin, whereby the hoppers diverge downward toward the circumference, so that the cement in sliding down the hopper would, as it were, spread itself around to the edges. That seemed to help considerably against the wedging proclivity of the cement at the top of the hopper. Mr. Heidenreich.

MR. F. L. WILLIAMSON.—In connection with the storage of cement it might be interesting to note that at our plant, cement is being stored in circular tanks, in interspace tanks and also in square tanks, all of reinforced concrete construction. The storage tanks are 80 ft. deep and 30 ft. in diameter. The interspace tanks, of course, are of corresponding size and three square tanks are 8 ft. square and 64 ft. deep. They have been in use for two years and have proven satisfactory. The strain Mr. Kinney speaks of has no doubt been exerted many times and with no damaging effects. Mr. Williamson.

REPORT OF THE COMMITTEE ON MEASURING CONCRETE.

Engineers and contractors for reinforced concrete structures have long felt the need of some standard method of measuring quantities in contracts for concrete work. This is especially the case in drawing up and adjusting unit price contracts. At present, although most engineers and contractors measure and estimate on the same general principles, they nearly all differ in the details of their work, with the result that disputes occur and sometimes serious loss is occasioned or injustice done. A feeling exists that some of the principles or fundamental rules of measurement are founded on a wrong basis and need careful consideration and revision. With this object in view this Committee was appointed last year to consider the matter and now submits its report.

In submitting these Proposed Standard Methods the Committee wishes to call attention to the purpose for which they are intended and the principles by which they were guided in framing them.

The *purpose* is, *first*, to establish a correct method for awarding unit price contracts and for measuring up work performed under the same; *second*, to inform concrete contractors and engineers of the best methods of estimating their work and working up unit costs; *third*, if the above two objects are attained there is likely to be greater uniformity in published cost data.

The *principles* which have guided the Committee in drawing up these rules are of great importance and are as follows:

First.—All work shall be measured net as fixed or placed in the building or structure, and therefore material cut to waste, voids, temporary work, etc., shall not be allowed for in measurement but in price.

Second.—In no case shall non-existent material be measured to pay for extra labor in different parts, but such difficult or expensive parts or extra labor shall be separately measured and described.

Third.—That all the chief items of labor and material entering into the cost of concrete work shall be separately measured and described by units that correctly represent the labor and material involved.

It follows from the first and second of these principles that all work shall be measured as it stands, and that forms or extra labor in placing concrete work shall not be allowed for by doubling or trebling the measurements of concrete, and that laps in reinforcement should be measured.

The acceptance of the third principle involves the recognition of the several items entering into the cost of a mass of concrete as separate and distinct operations, viz:

(1) Concrete mixing and placing.
(2) Forms.
(3) Reinforcement.
(4) Surface finish.

The separation of forms from concrete is not in opposition to the first principle of omitting incidental work, as forms should be considered as an item of labor. The labor of supporting wet concrete by means of forms is entirely distinct from the labor of mixing and placing concrete and is done by different men at a higher rate of pay. It is a distinct operation in the progress of the work and not a labor incidental only to the placing of the concrete.

The committee realizes that the proposed methods if generally adopted will be used and administered by inspectors, superintendents and foremen, as well as by engineers, architects and contractors. There are some who will search for opportunities for taking unfair advantages of the methods for their own profit while many will catch the spirit and use them fairly. It is, therefore, very necessary that care should be taken to provide no opportunity for fraud or unfair dealing.

In framing methods of measurement the Committee has felt that it is not necessary to give instructions as to *how* to measure, but only as to *what* units should be used in measuring and what items should be included or left out of such measurements. For instance, it is necessary to say whether I-beams shall be deducted or not deducted from the mass of concrete work, but it is not

necessary to give a method as to how to measure the irregular thickness of a tunnel lining or of the backing to a masonry wall. The latter are matters of mensuration and outside the scope of this report.

The Committee has endeavored to keep to methods of measurement only as distinct from methods of cost, and in general no instructions are given as to the way to fix the price or cost of any item, or as to how to make up any prices for work measured under these methods. For instance, the cost of plant is added by some contractors to the per yard cost of concrete while by others it is taken as a lump sum at the end of their estimate Nothing is said about this and in the committee's judgment should not be.

The Committee wishes to lay very special emphasis upon the classification of concrete, forms, reinforcement and surface finish into separate items. It is realized that especially in the case of forms a radical departure from the present method of measuring units is recommended, but after careful consideration the Committee feels convinced that the present method of including forms and finish and sometimes steel in the cubic measurement of monolithic concrete is fundamentally wrong and should be altered. A few instances will be cited to make this point clear.

Supposing a contractor takes a contract for a building with unit prices per cubic foot for concrete in floor, columns and foundations, the said unit to include the cost of forms and granolithic. If the engineer decides to reduce the thickness of the floor slab from 5 to 4 in. the contractor has to put in just as much form work, but is paid for only 80 per cent of the form work he estimated upon. If the building was increased from two stories to three stories in height and floors were finished in granolithic he would have to finish twice as many floors with granolithic as he estimated. Or vice versa, suppose a wall shown 8 in. thick is increased to 12 in. thick, the owner has to pay for 50 per cent more forms and surface finish than before, although the amount done is the same. In a dispute recently settled in the courts a contract provided for a certain price per cubic yard being paid for concrete foundations, this price covering the cost of concrete, forms and steel. In the execution of the work it was found necessary to make the footings deeper than shown, but most of the steel was omitted. The contractor claimed that though the steel was omitted his contract

price per yard of concrete was the one he was entitled to be paid on even if some of the steel was left out. The court upheld his view and he received judgment accordingly.

These instances are typical of the gambling nature of the present system of letting and taking contracts for concrete at an inclusive unit price per cubic foot, and the Committee feels confident that if the methods suggested are adopted they will be of real value to the community, eliminating such uncertainties and inequalities. Very few contractors in the present day like to take these gambling contracts.

In other respects the Committee has endeavored to conform as far as possible to the general practice in this country and desires to make it standard.

Some of the detailed points involved in the methods submitted will now be considered.

I. MONOLITHIC CONCRETE.

(a) *Concrete.*

The rule stating that concrete in different parts of the building or structures shall be measured and described according to its accessibility and location and the purpose of the work would indicate that there should be separate measurements appearing in the bill of quantities for:

Concrete in footings.
Concrete in columns.
Concrete in floors, beams and girders.
Concrete in paving.
Concrete in basement walls.
Concrete in curtain walls and partitions.

And so on according to the nature of the work.

Concrete in mass foundations.
Concrete in abutments.
Concrete in arch ribs.
Concrete in spandrel walls.
Concrete in bridge floors and beams.
Concrete in parapet walls and cornices.

And so on according to the nature of the work.

In the event of any of these items, such as columns, footings, etc., being of different mix on different floors or places, these also would be measured separately (see Appendix I).

The matter of placing bolts, inserts, pipes, etc., in the concrete was considered. It is not covered by any rule, as the Committee held that it was not strictly a concrete item. The placing of such items should be taken by the number or the lineal foot as the case may be and not allowed for in the price of concrete or forms. The cost of placing is part of the cost of the item itself, and as such would come under the heading of plumbing, steel work and so on.

(*b*) *Forms.*

After careful consideration the Committee feels that the square foot of the surface supported is the correct unit for the measurement of forms. In their judgment the item which should be measured and paid for is the operation of supporting the wet concrete until it is set. This item is practically an item of labor although material enters into the cost of same.

It is the practice of some firms to estimate forms by the amount of lumber used, but this is not a correct unit of measure, not only on the theoretical ground stated above, but on the practical ground that no two contractors would use the same amount of lumber in erecting a piece of form work, and if lumber on unit price contracts were measured and paid for by the board foot there would be an incentive to a contractor to use more lumber than was necessary to do the work.

It is necessary to give a definite unit which everyone knows how to measure. It is not possible to determine beforehand just how much lumber will be used in any piece of form work. It is not possible to accurately measure the amount of lumber used during the work, and it is not possible to take this amount from the lumber bills because all lumber bought on a job does not go into forms, some of it going into temporary buildings, sheeting and shoring and many other necessary parts. For instance, when the sheet piling of trenches is done, the lumber often goes into forms, and of 1,000 board feet of roofers delivered, half may go to the forms and half to temporary buildings. Further, much lumber is used two and three times. Other parts

are used but once only and some pieces are sawed off as waste and not used at all, and if a rule were to be framed that would measure lumber, how should all these difficulties be provided for.

Paying for lumber on a unit price contract gives to an unscrupulous man an opportunity to claim a higher payment from the owner, or in the case of cost data, for a foreman to deceive his employers as to the cost of the form work he is doing. A superintendent can put in twice as much bracing and posts as he actually needs and make his costs look low, and if the contractor is paid by the board foot he also would receive payment for unnecessary work.

In contrast to these difficulties the square foot measurement of form work does not alter and is easily and quickly measured; and further, even those who estimate by the board foot have to determine the number of square feet to be supported before computing the number of board feet to be required in the building.

In the measurement of floor forms the sides of beams are added to the measurement of under side of slab and beams, although some contractors take a flat measurement of floor surface. The latter method would work out unjustly in the case of a change being made in the depth of the beams, such change working to the detriment of either the owner or the contractor, while if the method that the Committee recommend is adopted such a change will adjust itself.

Some discussion may be raised on the omission of any allowance for angle fillets to columns and girders, but the Committee as practical concrete men realize that such items are a very small part of the cost of forms and it is not a usual practice on the part of contractors to estimate them separately. They believe that to measure angle fillet by the lineal foot, as has been suggested, would prove a possible source of misunderstanding in the carrying out of a contract.

The separation of forms to floors, columns, footings, etc., follows the lines laid down for the separation of concrete items.

In the case of forms to the concrete walls poured as backing to granite or other facing, the correct interpretation of the rules would be to measure forms to one face only, the stone facing doing the work of forms on the other side.

(*c*) *Reinforcement.*

Some may question the correctness of making an allowance for laps or passings in steel rods or fabric for reinforcement. The Committee after careful consideration felt that these ought to be allowed. Laps are for two purposes, one as in beam steel to take up negative moment, and the other to provide sufficient bond to take up the tensile strength in two bars where one bar cannot be obtained long enough for the purpose. In the first case no question would arise as to measuring the full length of each rod; in the second, we think that as the specification usually specifies the lap to be a certain number of diameters, usually forty or fifty, there would be little difficulty on this head.

There is, of course, an opportunity for unfair dealing by a contractor in putting in steel in short lengths. A rule not allowing lap would sometimes work hardship the other way if an inspector insisted that steel should go in in short lengths instead of long, and then only pay for a rod the net length of the building. An example would be in a floor of a building laid out in 10 ft. bays where rods could be put in one length over one, two or three bays as desired.

The Committee feels that the rule they have formulated should stand. It is in accordance with the first fundamental principle laid down and all work should be measured net as fixed in place. They believe that any engineer would use reasonable discretion in refusing to measure laps if it was apparent that too many had been made for an unfair purpose.

III. STRUCTURAL CAST CONCRETE.

The use of structural cast concrete is growing rapidly, is of an entirely different nature than monolithic concrete, and the Committee has endeavored to treat this part of the subject in a way similar to which structural steel is measured and estimated.

It is, therefore, not suggested that forms be measured separately, but recommended that erection should be separate from making and that the unit of weight be the correct unit for measuring erection rather than the unit of volume.

In measuring or computing erection quantities, it is recommended that an arbitrary weight of 150 lbs. per cu. ft. be adopted

(similar to the rule for steel). This method will save the expense of weighing each piece and a good deal of dispute. This rule would not apply to cinder concrete, for which another weight should be agreed upon.

In buildings built partly of cast concrete and partly of monolithic concrete, monolithic concrete would be measured under the rules laid down for same and the cast concrete under the structural cast concrete rules.

The Committee doesnot recommend that grouting be measured. This is looked upon as a small labor item incidental to the erection of the concrete members and similar in character to riveting on structural steel work or mortar in laying cut stone.

IV. CAST CONCRETE TRIM AND ORNAMENTAL WORK.

Cast concrete trim and ornamental work is more nearly akin to cut stone work than to any other trade. The Committee has followed the custom of the cut stone trade and based the rules on stone mason rules, viz: to measure the smallest rectangular solid out of which a piece can be taken for any piece of trim, instead of measuring the net volume of the finished block.

Surfacing is generally done in the mold and therefore should not be separated.

Temperature reinforcement is a very small part of the cost and for that reason is not separated.

Trim serving any structural purpose such as lintels with reinforcement in same to take tensile strength should be classified as structural cast concrete and measured accordingly.

As the stone mason's unit of erection is the same as the unit by which he supplies the stone, the Committee does not suggest that erection be separated from making.

The Committee has not at present drafted rules covering plastering, waterproofing, concrete blocks, concrete piles, etc., except in so far as they are covered by the general rules governing surface finish, concrete trim and structural cast concrete.

Plastering is understood to be any surface coating of cement or lime mixed with fine aggregates on soffits or vertical surfaces which is put on by hand without the use of forms.

The rules of structural cast concrete would apply to concrete piles cast and driven, except the driving is usually measured by the lineal foot and not by the pound, and the Committee does not suggest that this practice should be altered.

Respectfully submitted by the Committee on Measuring Concrete.

ROBERT A. CUMMINGS, *Chairman.*
L. H. ALLEN,
CHAS. DERLETH, JR.,
H. H. FOX,
THOS. M. VINTON.

APPENDIX I.

BILL OF QUANTITIES FOR A BUILDING.

Measured by Proposed Methods.

Concrete 1 : 2½ : 5.	Footings		500	cu. yds
	Basement walls		220	"
	Base to paving		90	"
Concrete 1 : 2 : 4.	Floors and beams		2,100	"
	Columns		650	"
	Partitions and curtain walls		200	"
	Walls, roof and pent house		30	"
	Cornice and parapet		100	"
Concrete 1 : 1½ : 3.	Columns		110	"
Cinder Concrete 1 : 3 : 6.	Fill between screeds (2 in. thick)		225	"
Forms.	Footings		1,000	sq. ft.
	Floor slabs and beams		15,500	"
	Columns		3,500	"
	Curtain walls and partitions		3,350	"
	Pent house		900	"
	Basement walls		3,000	"
	Parapet and back of cornice		1,200	"
	Face of cornice 36 in. girth		400	lin. ft.
Reinforcement.	Plain round bars, including cutting, wiring, placing	1¼ in. dia.	5,000	lbs.
		1 "	7,000	"
		⅞ "	12,000	"
		½ "	24,500	"
	Plain round bars, including cutting, wiring, placing, but also including fabricating	1½ "	8,000	"
		1 "	17,800	"
		½ "	28,500	"
		⅜ "	6,000	"
	Square twisted bars, including cutting, wiring and placing.	1 x 1 "	3,000	"
		⅞ x ⅞ "	8,000	"
		½ x ½ "	4,000	"
Granolithic Finish.	1 in. thick, laid integral with paving		5,000	sq. ft.
	1 in. thick, laid on floors after concrete has set		15,000	"
	On cornice and parapet		600	"
	On window sills		1,200	"
Picked face to concrete surface			2,000	"
Rubbed face and cement wash one coat on concrete surface			3,000	"

APPENDIX II.

BILL OF QUANTITIES FOR A BRIDGE.

Measured by Proposed Methods.

Concrete 1 : 3 : 6.	Cyclopean masonry with 30 per cent rock in foundation	2,000	cu. yds
	Abutments	1,500	"
	Piers	1,000	"
	Abutments (upper part)	500	"
	Piers (upper part)	500	"
	Arch ribs	1,200	"
	Columns above same	150	"
	Spandrel walls and wing walls	250	"
	Bridge floor and beams	600	"
Forms	Abutments (below finished grade)	12,000	sq. ft.
	Abutments (above finished grade)	6,000	"
	Piers (below finished grade)	10,000	"
	Piers (above finished grade)	8,000	"
	Arch ribs	15,500	"
	Columns	1,200	"
	Spandrel walls and wing walls	5,000	"
	Bridge floor and beams	12,000	"
	Parapet wall	2,500	"
	Coping of same	300	lin. ft.
Reinforcement.	Plain round bars 1¼ in.	30,000	lbs.
	" " " 1 "	50,000	"
	" " " ¾ "	75,500	"
	" " " ½ "	12,000	"
	" " " ⅜ "	10,000	"
Crandalled Finish.	Piers and parapet	2,500	sq. ft.
Rubbed Finish.	Arch ribs (face and soffits)	40,000	"
Granolithic Finish.	1 in. thick to sidewalks to bridge floor laid integral with slab	2,000	"
	Curb and gutter of same	300	lin. ft.
Sidewalk.	To approaches with 4 in. base 1 : 2½ : 5 and 1 in. top and includes cinder foundation	800	"
	Curb and gutter to last, curb 10 in. high, gutter 12 in. wide including forms and finish done in one operation	200	"
	Extra for rounded corners to same 3 ft. girth	4	
Cast Concrete.	Balusters 8 in. x 8 in. x 2 ft. high, including forms and steel and setting in place	50	
	Coping to same 18 in. x 9 in. with molded edges, including forms, steel and finish and setting in place	200	lin. ft.

PROPOSED STANDARD METHODS FOR THE MEASUREMENT OF CONCRETE WORK.*

The following divisions are recognized as separate and distinct items in the construction of concrete work for which separate modes of measurement are necessary.

I. Monolithic Concrete:

(*a*) Concrete.
(*b*) Forms.
(*c*) Reinforcement.
(*d*) Surface Finish.

II. Sidewalks.

III. Structural Cast Concrete:

(*a*) Concrete.
(*b*) Reinforcement.
(*c*) Erection.

IV. Cast Concrete Trim and Ornamental Work.

The following general rules shall govern the measurement of the above items (with the exceptions where specifically noted):

(*a*) All work shall be measured net as fixed or placed in the structure.

(*b*) In no case shall non-existent material be measured to cover extra labor.

(*c*) No allowance shall be made for waste, voids, or cutting.

I. Monolithic Concrete.

(*a*) *Concrete.*

1. The unit of measure for all concrete shall be the cubic foot.

2. In no case shall the measurement of concrete be held to include the forms.

* The proposed methods submitted by the Committee were discussed, referred back to the Committee and appear here as amended.—Ed.

3. All concrete shall be measured net as placed or poured in the structure.

4. In no case shall an excess measurement of concrete be taken to cover the cost of forms or extra labor in placing.

5. All openings and voids in concrete shall be deducted with the following exceptions:

(a) No deduction shall be made for reinforcement, I-beams, bolts, etc., embedded in concrete except where a unit has a sectional area of more than 1 sq. ft.

(b) No deduction shall be made for pipes or holes in concrete having a sectional area of less than 1 sq. ft.

(c) No deduction shall be made for chamfered, beveled or splayed angles to columns, beams and other work, except where such chamfer, bevel or splay is more than 4 in. wide measured across the diagonal surface.

6. Each class of concrete having a different proportion of cement, sand or aggregate shall be measured and described separately.

7. Concrete in the different members of a structure shall be measured and described separately according to the accessibility, location or purpose of the work.

8. Concrete with large stones and rocks embedded in same (cyclopean masonry) shall be measured as one item and described according to the richness of the mix and the percentage of rock in same.

9. Concrete in stairs shall be measured by the cubic foot and shall include surface finish when the mixture is the same throughout.

(b) *Forms.*

10. The unit of measure for form work shall be the square foot of actual area of the surface of the concrete in contact with the forms or false work.

11. Forms shall in every case be measured and described as a separate item and in no case shall the measurement of concrete be taken to include forms.

12. No deduction shall be made in measurement of surface of concrete supported by forms, because of forms being taken down and re-used two or three times in the course of construction.

13. The unit price for superficial measurement of forms shall be deemed to include the cost of struts, posts, bracing, bolts, wire ties, oiling, cleaning, and repairing forms.

14. No distinction shall be made between wood and metal forms.

15. Forms to different parts of a structure shall be measured and described separately according to the position in the structure, accessibility, purpose and character of the work involved.

16. No allowance shall be made for angle fillets or bevels to beams, columns, etc., but curved moldings shall be measured and described separately as hereinafter provided.

17. No deduction in measurement of forms shall be made for openings having an area of less than 25 sq. ft.

18. No deduction shall be made in floor forms for heads of columns of any shape.

19. No deduction shall be made in column and girder forms for ends of girders, cross beams, etc.

20. No allowance shall be made for hand-holes in column forms for clearing out rubbish.

21. The measurement of column forms shall be the girth of the four sides or circumference multiplied by the height from the floor surface to the under side of floor slab above.

22. Forms to octagonal, hexagonal and circular columns shall be measured and described separately from forms to square columns.

23. Caps and bases to columns and other ornamental work shall be measured by number and fully described by overall dimensions.

24. The measurement of beam forms shall be the net length between columns multiplied by the sum of the breadth and twice the depth below the slab, except for beams at edge of floor or around openings which shall have the thickness of floor added to the sum of the breadth and twice the depth.

25. Wall forms shall be measured for both sides of concrete wall.

26. Allowance shall be made by number for pockets left for future beams.

27. Moldings in form work shall be measured by the lineal foot.

28. Forms to circular work shall always be measured separately from forms to straight work.

29. No measurement or allowance shall be made for construction joints in slabs, beams or arch ribs, to stop the day's concreting.

30. Construction joints or expansion joints to dams and other large masses of concrete shall be measured by the square foot as they occur.

31. Forms to cornices shall be measured by the lineal foot and the girth stated. (The term girth shall be taken to mean the total width of all curved and straight surfaces touched by the forms.) Plain forms to back of cornice to be measured separately.

32. Forms to window sills, copings and similar work shall be measured by the lineal foot.

33. Forms to the upper side of sloping slabs such as saw tooth roofs shall be measured whenever the slope of such slab with the horizontal exceeds an angle of 25 degrees.

34. Forms to the under side of stairs shall be measured by the superficial foot.

(*a*) Forms to the front edge of the stairs shall be measured by the lineal foot.

(*b*) Forms to the ends of steps shall be measured by number.

(*c*) *Reinforcement.*

35. The unit of measure of reinforcement shall be the weight in pounds.

36. The weight shall be calculated on the basis of a square rod 1 in. x 1 in. x 12 in., weighing 3.4 lb.

37. Steel rods for reinforcement shall be measured as the net weight placed in the building.

38. Deformed bars shall be measured separately from plain.

39. No allowance shall be made for rolling margin.

40. No allowance shall be made for cutting or waste.

41. No allowance shall be made for wire ties, spacers, etc.

42. No separation shall be made according to accessibility, location and purpose of reinforcement except in special cases.

43. In measuring reinforcement the rods shall be measured by the lineal foot as laid. All laps shall be allowed for.

44. The rods of each different size shall be measured and described separately.

45. Bent bars shall be measured separately from straight bars.

46. Pipe sleeves, turnbuckles, clamps, threaded ends, nuts and other forms of mechanical bond shall be measured separately by number and size and allowed for in addition.

47. Wire cloth, expanded metal and other steel fabrics sold in sheets or rolls shall be measured and described by the square foot. The size of mesh and weight per square foot of steel in tension shall be stated. No allowance shall be made for waste, cutting, etc., but all laps shall be measured and allowed for.

(*d*) *Surface Finish.*

48. The unit of measure for finish of concrete surfaces shall be the square foot. Finish shall always be measured and described separately.

49. No measurement or allowance shall be made for going over concrete work after removal of forms and patching up voids and stone pockets, removing fins, etc.

50. Granolithic finish shall be measured by the square foot and shall include all labor and materials for the thickness specified.

51. Finish laid integral with the slab shall be measured separately from finish laid after the slab has set.

52. No allowance shall be made for protection of finish with sawdust, sand or tenting.

53. Grooved surfaces, gutters, curbing, etc., shall be measured separately from plain granolithic and shall be measured by the square foot or lineal foot as the case may require.

54. The following shall be measured by the square foot:

Cement wash. (State how many coats.)
Rubbing with carborundum.
Scrubbing with wire brushes.
Tooling.
Picking.
Plastering.
Etc.

II. SIDEWALKS AND PAVEMENTS.

55. Sidewalks and pavements shall be measured by the square foot.

56. The one measurement shall include concrete, finish, lining in squares and cinder or stone foundations.

57. Curbs and curb and gutter work shall be measured by the lineal foot and separated according to character and size, and shall include foundations, forms, finish and cost of special tools if any.

58. In measuring curbs the full height and width or thickness of same shall be taken, but the measurement of sidewalks shall also be taken the extreme width of horizontal surface.

59. Circular corners to curbs and gutters shall be measured separately by number, stating radius and length measured on the curve.

60. Vault lights shall be measured by the square foot, the measurement to include glass, forms, steel and finish. Beams under vault lights shall be measured by the lineal foot. In measuring vault lights the measurement shall go at least 4 in. beyond the outside line of the glass in each direction.

III. STRUCTURAL CAST CONCRETE.

(*a*) *Concrete.*

61. The term structural cast concrete is taken to include unit construction by the various systems.

62. The unit of measurement for structural cast concrete shall be the cubic foot, and shall be measured net as provided for monolithic concrete.

63. The various members shall be measured on the ground before erection.

64. No measurement shall be taken of forms.

(*b*) *Reinforcement.*

65. Reinforcement shall be measured separately as provided in Paragraphs 35 to 47, inclusive.

(*c*) *Erection.*

66. The unit of measure for the erection of structural concrete shall be the weight of the finished member in pounds.

67. In measuring the erection of structural cast concrete having a crushed stone or gravel aggregate, the concrete shall be assumed to weigh 150 lb. per cu. ft.

68. No measurement shall be taken of the grouting in structural cast concrete. It shall be deemed to be covered in the price of erection.

IV. CAST CONCRETE TRIM AND ORNAMENTAL WORK.

69. Cast concrete trim shall be measured by the cubic foot, but the measurement shall be the smallest rectangular solid that will contain the piece measured and not its actual content.

70. No allowance shall be made for forms.

71. No allowance shall be made for reinforcement in trim and ornamental work.

72. No allowance shall be made for surface finish in trim and ornamental work.

73. Circular work shall be measured separately from other work.

74. Mitre blocks and end blocks for cornices, etc., shall be measured separately from straight molded work.

75. Vases, seats, pedestals, balusters and other similar items shall be measured by number and description with overall dimensions.

CONCRETE RETAINING WALLS.

By J. M. Meade.*

Concrete walls are now mostly built of two types, viz., plain and reinforced. They are very popular for track elevation, depression and dike work, especially where ground is valuable and they replace earth dikes to economize space, etc.

PLAIN MONOLITHIC CONCRETE RETAINING WALLS.

The plain concrete walls are designed for what is known as gravity section, being heavy enough so that their weight and stability will stop them from overturning. The best authorities figure the width of base of such walls as 0.45 up to 0.65 of their height, varying the width according to circumstances. Where a retaining wall of this type is built up close to the end of ties, as on an elevated road, it becomes a surcharged wall. When designed as 0.45 I have seen them fail by pushing over and would recommend not less than 0.65 of the height for a surcharged wall, finishing 18 in. wide at the top. The common practice in railroad work of using arbitrary ratios of width of base to height of walls tends to cause a neglect of the study of the proper distribution of the pressure on the foundation and it seems to be difficult to get away from such practice. It is a well-known fact that movement from the original alignment, due to unequal settlement, is the most common cause of failures or defects. The writer has in mind some flagrant cases of this kind in the City of Chicago that have caused the owners a very heavy expense. This question is one of great importance and each particular case should be carefully investigated and studied, so the amount and distribution of the pressure on the foundation may be accurately determined.

Many walls of poor design have come to the attention of the writer, there being entirely overlooked, due to a lack of analysis of the design, the most effective section and minimum amount of material for an economic design.

* Engineer, Eastern Lines, Atchison, Topeka and Santa Fé Railway, Topeka, Kan.

In constructing retaining walls, it is of the greatest importance that serious thought be given to the matter of earth filling and embankments behind the walls. The drainage is quite easily accomplished by filling or placing close up to the back of the wall some open or porous material, such as crushed or refuse stone, large size gravel, brick bats, etc.; cinders will also do considerable good. Weep holes should be placed in the wall of 3 or 4 in. drain

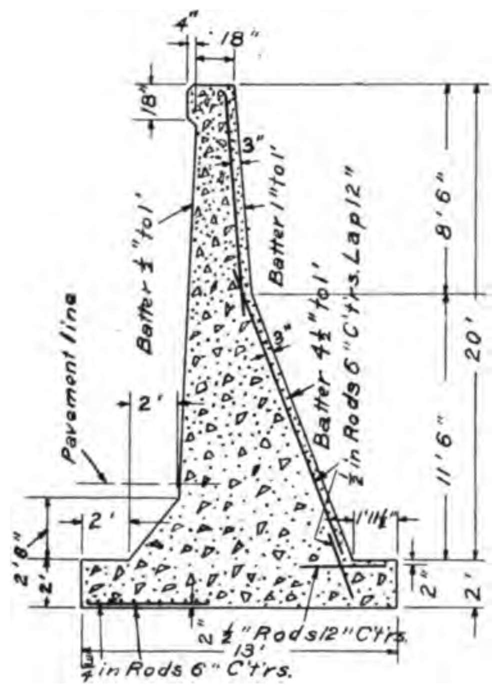


FIG. 1.—REINFORCED CONCRETE TYPE OF RETAINING WALL AS USED IN RAILROAD WORK.

tile, vitrified, about 15 or 20 ft. apart, according to conditions, extending the blind drains to a point near the top of the wall if circumstances seem to warrant. It is also quite important in plain concrete retaining walls, to use expansion joints about 30 ft. apart, of the dovetail pattern. If this is not done temperature stresses will crack the wall. Such expansion joints are a good investment.

REINFORCED CONCRETE RETAINING WALLS.

Reinforced concrete retaining walls consist of either a thin vertical plane attached to a horizontal base and well braced by counterforts on the back and buttresses in front or they may be designed as cantilevers, in which case the wall is connected to a wide base resembling an inverted T.

It has been found in actual practice that reinforced concrete walls are more economical than the plain monolithic gravity walls, as the material in the latter type cannot be fully utilized for the reason that the section must be made heavy enough so that the dead weight and size prevent overturning. On the other hand, in reinforced concrete walls a part of the retained material is used to prevent overturning and the wall only need be made strong enough to withstand the moments and shears due to the earth pressure. The wall is lighter and exerts less pressure on the ground, which, with the opportunity of extending the base of the wall, often enables the builder to use ordinary foundations instead of piles.

Reinforced walls allow the use of a more scientific design than the gravity walls and have been known to be more reliable than the plain concrete. It is quite common practice to make the base of these reinforced or cantilever type of walls about 0.60 of their height and then reduce in size about as shown by Fig. 1.

DISCUSSION.

Mr. Willis Whited.—I would like to ask Mr. Meade if the long dike at Topeka is built open, exposed to the action of floating ice and, so far as he can judge by the experience so far had, whether it is necessary to thicken the wall to resist the impact of floating ice. We have some work in Pennsylvania on some of our highways where it is necessary to use a very long dike which will be exposed almost to the top in flood time and there would be a good deal of impact from floating ice against it. Mr. Whited.

Mr. John M. Meade.—In a great many instances the reinforced concrete wall has proven very economical in cities. At Topeka, where the floods of 1903 were so disastrous, the river was diked on one side with an earth dike and on the other side with a reinforced concrete wall, which was largely planned on account of the valuable right of way. It is one of the longest concrete dikes in this country, being about a mile in length. The wall was finished late last fall and of course we have not had any tests yet. It is 18 in. thick at the top. The river has only been up to above normal or high water a few feet, and the wall is set back well from the bank, in some places 30 or 40 ft., so it would take a big flood to get above the banks and reach this wall. With a heavily reinforced wall the ice would shear off so rapidly, going parallel with the wall, as to cause little pounding. Mr. Meade.

This work was especially watched with reference to its cost of construction, etc., being a new departure in dikes. Another departure was the use of Joplin chatts. There have been some misgivings as to the reliability, for heavy work, of chatts, a by-product in the manufacture of zinc. The chatts can be had for about the cost of loading, which is about 15 cents a cu. yd. at Joplin and of course railroads do not figure the freight. I had occasion to look at a pier built over the Chikaskia River in the southwestern part of Kansas on a line used jointly by the Santa Fé and the Frisco Railroads. A stone pier had failed about eight years ago and was re-built with Joplin chatts. There was a dam below the bridge, so that the pier has been in water from 6 to 8 ft. deep all the time. I was very favorably impressed with the results of the use of chatts at that place.

REINFORCED CONCRETE PILES.

By Robert A. Cummings.*

It is well established that concrete piles are a satisfactory substitute for wooden piles in increasing the stability of foundations, whether they are of the *cast-and-driven* type or the *cast-in-place* type. The *cast-and-driven* type is made of reinforced concrete, molded to the desired shape and cured before being driven. The *cast-in-place* type is made by forming a hole in the ground and filling it with concrete.

The history of the *cast-and-driven* type is intimately connected with the development of reinforced concrete and in the same manner has been embarrassed by patent litigation. In March, 1907, a final decision was reached in the British High Courts of Justice that the fundamental idea of the reinforced concrete pile was covered by Brannon's patent of 1871 and that subsequent improvements must be limited to the details of reinforcing.

At the Chicago Convention of this Association in 1909, the writer described certain tests and methods of reinforcing for increasing the unit value of concrete in compression. It is the application of one of these methods and other practical improvements that form the subject of this paper.

It is nearly ten years since the writer began the design and manufacture of concrete piles. During this period, by a process of elimination, an efficient method of reinforcing has been developed, which is correct in design and economical in cost.

METHOD OF REINFORCEMENT.

All piles of the *cast-and-driven* type must be reinforced with longitudinal rods, because the pile is hoisted and handled by a line from the pile driver which is fastened at or near the butt. Consequently, the pile must sustain its own dead weight while being raised, as well as shocks and impact against obstacles,

*Consulting Engineer, Pittsburgh, Pa.

before reaching its position in the leads of the pile-driving machine. With the point on the ground and the other end being elevated the pile must act and be designed as a beam supporting its own dead weight and shocks. The limiting proportions of depth or thickness of pile to the unsupported beam length produces heavy tensile and compressive stresses, with considerable deflection.

It has been observed that when handling a pile of this type, it rarely fails in compression in the concrete, but cracks are usually discovered on the tension side. These cracks can be accounted for by the slipping of the usual longitudinal rods used in the concrete while the pile is being hoisted. While such cracks are not sufficiently serious to condemn the pile, they may affect the permanency of the reinforcement.

In order to overcome this defect all longitudinal rods should be anchored at the ends, those at the butt opposite each other being bent over into a loop and welded together, while those at the point are all brought together and electrically welded into one piece. Twisted and deformed rods are advantageous for longitudinal reinforcing, as the allowable bond stress is higher than for plain rods.

The uniform circumferential spacing of longitudinal rods is very important, because any side of the pile may be subjected to tensile stresses and whichever side is in tension there must be sufficient reinforcement in position to take the strain. The circumferential spacing of the longitudinal rods can be secured by means of a special spacing device placed at intervals of about 5 ft. throughout the length of the pile.

The hooping of concrete adds greatly to its ability to resist axial loads. Therefore, longitudinal reinforcing should have a helical wrapping of wire throughout the length of the pile, the pitch of which must not exceed 3 in. This wire wrapping will assist in taking care of diagonal stresses resulting from the handling of the pile.

Practical experience indicates that the butt end of the pile which receives the impact of the hammer should be especially reinforced. This has been done by means of a special reinforcement consisting of a unit cage of flat bands, held 2 in. on centers by a spacing bar for a distance of 2 ft. In the plane of each band a flat wire spiral is fastened to the cage. The embedment

of this unit cage in the butt of the pile forms a resilient cushion to receive the impact of the hammer. In no case, in the driving of thousands of piles with this cushion, has the butt itself been broken.

The general design of the pile is shown in Fig. 1.

METHODS OF DRIVING.

Piles of the *cast-and-driven* type are handled and driven by means of an ordinary pile driver. The ability to vary the fall of the drop hammer is of great utility in overcoming the variable resistances to be met with in the driving. It has been found

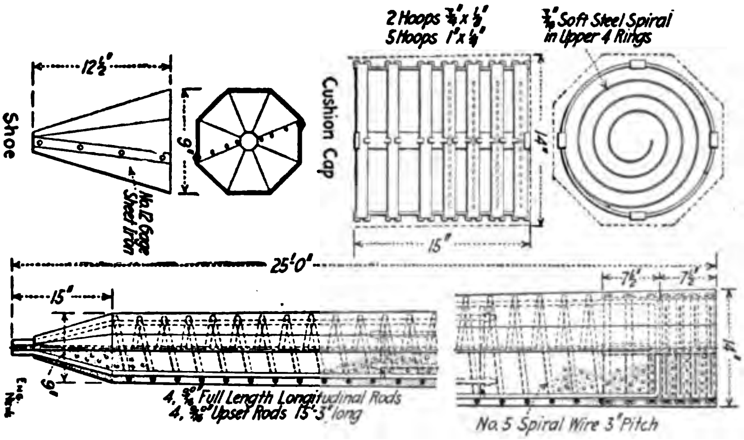


FIG. 1.—DESIGN OF REINFORCED CONCRETE PILE.

advisable to increase the weight of the ordinary drop hammer in ratio of weight of hammer to weight of pile of from 2 or 3 to 1, so that the weight of the drop hammer for driving the concrete piles will vary from 7000 to 12000 lb.

Steam hammers are not as efficient or desirable for driving concrete piles as are drop hammers. This was shown last fall on a contract when a test was made between a steam hammer and a heavy drop hammer, under the same conditions, using the same kind and size of concrete pile. The heavy drop hammer did not break a pile, whereas the steam hammer broke several below the cushion. Further, the steam hammer did not drive as many piles in a given time as did the drop hammer.

The following explanation is offered as it applies also to the driving of the heavy steel casings or core used in the making of the *cast-in-place* type of pile: The limited fall (3 ft.) and light ram (3000 lb.) of the steam hammer, while delivering twice as many blows per minute as the drop hammer, loses a large part of its energy in overcoming the proportionately heavier weight of the pile, steel casing or core.

The following analytical treatment by Mr. Barton H. Coffey, of New York, confirms and justifies the practice of the writer in using hammers as heavy as 12,000 lb. for driving concrete piles and it may be asserted with some feeling of confidence that if penetration is to be the gauge for measuring the supporting power of a pile, the ratio of the weight of the hammer to the weight of the pile or core that is driven must be taken into consideration.

The advantage of using a heavy hammer is evident from the following analysis:

Let M = Mass of hammer $= \frac{W}{g}$

" W = Weight of hammer.

" X = Weight or mass of pile in per cent of hammer.

" V = Velocity of hammer on striking pile.

" V_1 = Common velocity of hammer and pile.

Momentum of hammer $= MV$ (on striking pile.

" " " and pile at common velocity $= M\ (1+X)\ V_1$.

These are equal, provided no external force acts, which we will assume for the present is the case.

$$\text{Then } MV = M\ (1+X)\ V_1\text{; therefore } V_1 = \frac{V}{1+X} \ldots\ldots (a)$$

The energy in hammer on striking pile is

$$= \tfrac{1}{2} MV^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (b)$$

The energy in both hammer and pile at common velocity is

$$\tfrac{1}{2}M\ (1+X)\ V_1^2 \text{ which upon substituting } (a) \text{ becomes } \tfrac{1}{2}M\ \frac{V^2}{1+X} \ldots\ldots (c)$$

The difference between (b) and (c) represents the *loss of kinetic energy* in the system at point of common velocity or greatest compression. In other words, the percentage of the

original energy of hammer that has gone into compressing pile and hammer, distorting or crushing them and into heat, or broadly, internal work done upon each other by the striking bodies. Obviously, the smaller this percentage of internal work the less liability there is of crushing or distorting the pile. To resume

$$(c)-(b)=\tfrac{1}{2}\,MV^2-\tfrac{1}{2}\,M\,\frac{V^2}{1+X}=\tfrac{1}{2}\,MV^2\left(1-\frac{1}{1+X}\right)\ldots\ldots(d)$$

This equals the internal work.

The following table gives internal work for various values of X in per cent of the total kinetic energy of the hammer.

X	Internal Work.
0.25	20 per cent
0.50	33 "
0.75	43 "
1.00	50 "
1.50	60 "
2.00	67 "
3.00	75 "

If the hammer weighs 4 tons and the pile 1 ton, there will be 20 per cent internal work at maximum compression, whereas if the pile weighs 8 tons the internal work will be 67 per cent. There is an external force acting against the pile, *i.e.*, the frictional and displacement resistances of the earth.

Two extreme cases may be assumed limiting all others.

1. There is no external force. There the internal work will be simply that necessary to overcome the inertia of the pile and put it in motion. In this case the table is rigorously accurate.

2. The external force is great enough to prevent any movement of the pile. In this case the entire kinetic energy of the hammer goes into the internal work and the relative weights of hammer and pile are immaterial.

All intermediate cases where movement occurs are a combination of (1) and (2), where obviously it is advantageous to employ a heavy hammer.

MANUFACTURE.

The procedure in constructing *cast-and-driven* piles commences with the preparation of the molding bed. This, of course, will vary with the site, but it is desirable to select a flat and convenient location near the place of driving. It is very important that the bed shall be stable, so that settlement due to the weight of the piles is avoided. Where the ground is soft and yielding, pine stakes 2 in. x 4 in. x 3 ft. long, pointed at the ends, are driven to a solid bearing. These stakes are located at intervals of about 4 ft. in each direction and the tops cut to a uniform level. Then, 4 x 4 in. pine sills are toe-nailed to the top of the stakes in longitudinal rows about 4 ft. on centers. Upon these sills a 2-in. solid wooden floor is placed, which forms the molding bed. It is desirable that this bed shall be uniformly level to receive the forms for the piles.

The forms are made of two pieces of 2 x 8-in. dressed pine, battened together and placed on edge to form the sides of the pile. The bevels or angles for tapered or octagonal piles are made by placing loose pieces of bevelled wooden strips at each corner of the form. The reinforcement is delivered on the work in factory-made-units, so that it can be placed in the forms at once. When a reinforcing unit is suspended and centered in position, the concrete of a wet consistency is deposited and carefully puddled. As soon as the concrete of the pile has solidified, the forms are stripped and used for making other piles. The number of forms required will vary with the quantity of piles to be made and the prospective salvage in the lumber.

The curing of concrete in the normal manner delays the driving of the piles for a period of not less than 3 weeks, although a greater length of time is desirable, especially in cool and damp weather. Therefore, unless a stock of cured piles is always on hand, it frequently occurs that this type of pile cannot be used at all and resort is had to the use of the *cast-in-place* type. This practice is open to question on account of the inability of plain concrete to resist even moderate tension. In fact, it is almost axiomatic that all concrete piles must be reinforced. Every concrete pile is subjected to strains that induce very serious tensile stresses in the pile. Such stresses may result from superimposed loads or a lateral strain from the soil. Further, the

making and storing of concrete piles for use at any time necessitates a large financial investment.

In order to avoid the above-mentioned objections, the writer has adopted the method of steam curing of *cast-and-driven* piles. This enables such piles to be made and driven within 3 or 4 days and places the speed of driving on the same basis as that of the *cast-in-place* type.

The means used for steam curing will vary with circumstances, location of the work, speed required for delivery of the piles, the number needed, etc. During the past winter the writer has used concrete piles made and driven within 10 days, and, as a result of this experience, confidently recommends that such piles can be made, cured and driven immediately.

On the work with which the writer was connected, the piles

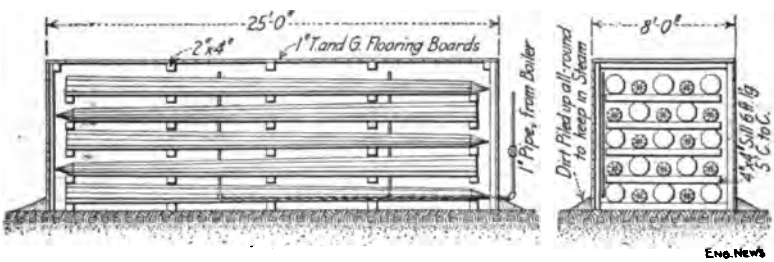


FIG. 2.—SHED FOR STEAM-CURING PILES.

were allowed to set in a normal manner for 5 or 6 days—and were then gently hoisted from the molding bed by a derrick, using an equalizing spreader and bridle, the chains of the bridle being fastened so that the pile was balanced. They were then placed in stacks of 25 or 30 and separated from one another by wooden blocks, particular attention being given to securing a solid bearing for each pile.

A light wooden shed, practically steam tight, was built entirely around the stack of piles, Fig. 2. The steam was conducted direct from a boiler through a 1-in. pipe to 3 branch openings inside the shed. The steam pipe valve was opened and the piles were exposed to live steam for 2 or 3 days, when they were found to be ready for driving. On being first exposed to steam, the moisture condensed on the surface of the piles and remained until absorbed by the concrete when the temperature of

the steam was reached. This steam treatment should be distinguished from heat applied indirectly or baking, in accelerating the set and hardening the concrete. The writer sees no reason why boiling water should not be used for the same purpose if the conditions are favorable for adopting this method. Precaution should be exercised in making sure that the concrete has solidified and that it has received its initial set, before exposure to steam treatment.

The prospective field opened up by the steam treatment for the rapid curing of concrete seems to the writer to solve the difficulty incidental to the present methods of procedure in the construction of all classes of concrete structures.

Attention is directed to the publication of the tests of the Structural Materials Laboratory of the United States Geological Survey, wherein it is conclusively shown "that a compressive strength considerably (in some cases over 100 per cent) in excess of that obtained normally after ageing for six months, may be obtained in two days by using steam pressure for curing mortar."

Practicing engineers will have little difficulty in modifying the writer's methods and using improved schemes for quick curing of the piles with steam; and in this connection it may be of interest to state that the writer has already under way the construction of steel molds and appliances for the steam treatment of concrete piles.

A list of references on Concrete Piles to 1908 is given in the appendix.

APPENDIX.

LIST OF REFERENCES ON CONCRETE PILES.

CONCRETE PILES FOR SANDY GROUND. *Engineering News*, v. 49, p. 275. (March 26, 1903.) (Description of the Raymond system of sinking piles with water jet. Illustrated.) (1 column.)

A CONCRETE-STEEL PILE FOUNDATION. *Engineering News*, v. 49, p. 173. (February 19, 1903.) (Description of piles used for the Court House, Berlin, Germany. Driven by steam hammers.) (1 column.)

CONCRETE PILE FOUNDATIONS, CARNEGIE PUBLIC LIBRARY, AURORA, ILL. *Engineering News*, v. 48, p. 495. (December 11, 1902.) (Description of the Raymond method of using steel core for making holes for concrete piles. Illustrated.) (½ column.)

A NEW SYSTEM OF CONCRETE PILE CONSTRUCTION. *Engineering News*, v. 45, p. 450. (June 20, 1901.) (Description of the trial of the Raymond steel core system, made at Chicago, May 16, 1901. Illustrated.) (1 column.)

CONCRETE PILE FOUNDATIONS. *Engineering News*, v. 46, p. 75. (August 1, 1903.) (Description of the foundations of the nine-story apartment house built for W. J. Bryson, Lake Avenue, Chicago. Holes bored by water-jet system before Raymond steel core was dropped. Material largely sand and quicksand.) (½ column.)

THE HENNEBIQUE SYSTEM OF ARMORED CONCRETE CONSTRUCTION. By Leopold Mensch. *Journal Association of Engineering Societies*, v. 29, p. 108. (September, 1902.) (Contains three pages on concrete-steel piles.)

Abstract of same. CONCRETE-STEEL PILES OF THE HENNEBIQUE SYSTEM. *Engineering Record*, v. 46, p. 618. (December 27, 1902.)

CONSTRUCTION IN CONCRETE AND REINFORCED CONCRETE. By F. C. Marsh, 1902. Minutes of Proceedings of the Institution of Civil Engineers, v. 149, p. 297. (Gives a short description of piles made after the Hennebique System.)

NEUERE BAUWESEN UND BAUWERKE IN BETON UND EISEN, NACH DEM STANDE BEI DER PARISER WELTAUSSTELLUNG 1900. Fritz v. Emperger, *Zeitschrift des Oesterreichischen Ingenieur- und Architekten-Vereines*, 53. Jahrgang, pp. 713 and 765. (October 25 and November 15, 1901.) (General description of concrete piles and their use.) (Illustrated. 7 pages.)

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CONCRETE PILES. *Railroad Gazette*, v. 34, p. 645. (August 15, 1902.) (Description of the Raymond System with illustrations from the J. I. Case Plow Works, Racine, Wis.) (1 column.)

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CONCRETE PILE FOUNDATIONS OF THE HALLENBECK BUILDING, NEW YORK. *Engineering Record*, v. 47, p. 377. (April 11, 1903.) (Piles sunk by means of water jet. Material, gravel and sand. Illustrated.) (1 page.)

QUAIMAUERN UND FUTTERMAUERN AUS BETON UND EISEN (SYSTEM HENNEBIQUE). *Zeitschrift des Oesterreichischen Ingenieur- und Architekten-Vereines*, 53. Jahrgang, p. 539. (August 9, 1900.) (Short descriptions of quay walls in Southampton, Paris and Nantes.) (Illustrated. 1½ pages.)

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PFAHLROSTKONSTRUKTIONEN IN BETON-EISEN. *Deutsche Bauzeitung*, 36. Jahrgang, p. 411. (August, 1901.) (Short description of use of concrete piles according to Hennebique system for a building for Holland-American line in Rotterdam.) (½ column.)

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RECENT DEVELOPMENTS IN PNEUMATIC FOUNDATIONS FOR BUILDINGS. By D. A. Usina, Associate American Society Civil Engineers. *Proceedings American Society of Civil Engineers*, v. 34, p. 220. (March, 1908.) (Contains two pages on the comparison of concrete piles and caissons for foundations of buildings.)

FOUNDATIONS; AN INFORMAL DISCUSSION AT THE ANNUAL CONVENTION, JULY 10, 1907. *Proceedings American Society Civil Engineers*, v. 33, p. 812, 816. (September, 1907.) (Contains some data on concrete piles.)

CONCRETE AND CONCRETE-STEEL. *Transactions American Society Civil Engineers*, v. 54, Pt. E., pp. 436, 461, 469, 548, 615. (International

Engineering Congress, St. Louis, 1904.) (Very short references to concrete piles.)

CYLINDRICAL FOUNDATIONS FOR A QUAY WALL IN THE HARBOR OF DELFZYL. By W. F. Druyvesteyn. *Transactions American Society Civil Engineers*, v. 54, Pt. E. (International Engineering Congress, St. Louis, 1904.) (Contains 1 page on concrete piles.)

USE OF REINFORCED CONCRETE IN BUILDINGS. By Frank C. Schmitz. *Proceedings Brooklyn Engineers' Club*, 1906, p. 67. Brooklyn, N. Y. 1907. (Contains a short table comparing the price of wooden and concrete piles.)

DRIVING CONCRETE PILES BELOW THE BATTERY TUNNEL, NEW YORK. *Engineering Record*, v. 55, p. 678. (June 8, 1907.) (The construction consists of a series of transverse pile bents at irregular intervals of about 50 feet supporting the undersides of the tubes for two lengths of about 600 feet each. In each bent there are two reinforced concrete piles 20 inches in diameter and about 7 feet apart on centers.)

GRADE CORRECTION AND PILE FOUNDATIONS IN THE EAST RIVER TUNNEL OF THE NEW YORK RAPID TRANSIT SUBWAY. *Engineering News*, v. 57, p. 718. (June 27, 1907.) (An illustrated description of the construction of the reinforced concrete piles.)

CONCRETE PILE FOUNDATIONS FOR A TOWER 700 FEET HIGH. *Engineering Record*, v. 55, p. 531. (April 27, 1900.) (The piles are of the Raymond concrete type and are made in the standard manner by first driving with a solid steel core a thin conical steel shell, which excludes water and sand and is filled with concrete after the core is withdrawn.)

STEAMSHIP TERMINAL WITH CONCRETE PILE PIERS AT BRUNSWICK, GA., ATLANTIC AND BIRMINGHAM RAILWAY. *Engineering News*, v. 56, p. 654. (December 20, 1906.) (Gives specifications for concrete piles, illustrations and method of construction.)

CEMENT PIERS. *Scientific American Supplement*, v. 63, p. 26241. (May 25, 1907.) (The cement cylinders for use in piers at San Francisco are made of three wooden piles enclosed in reinforced concrete.)

IMPROVED SYSTEM OF CONCRETE PILING. *Journal of the Franklin Institute*, v. 160, p. 455. (December, 1905.) (A report of a committee on the merits of the concrete pile invented by Frank Shuman, illustrated.)

THE SIMPLEX SYSTEM OF CONCRETE PILING. By Constantine Shuman. *Proceedings Engineers' Club of Philadelphia*, v. 22, p. 347. (October, 1905.) (Illustrated.)

CONCRETE PILES, DESCRIPTION OF THE METHODS OF MANUFACTURE AND USAGES OF THE TWO LEADING TYPES OF CONCRETE PILES WHICH ARE REPLACING THE WOODEN PRODUCTS. By David Lay. *Cement Age*, v. 2, p. 626. (February, 1906.)

RECONSTRUCTION OF THE ATLANTIC CITY "STEEL PIER" IN REINFORCED CONCRETE. *Engineering News*, v. 56, p. 90. (July 26, 1906.) (The reinforced concrete piles were molded on small pile platforms adjacent to the location of the piles in the piers; after hardening the piles were lifted from the platforms, set in position and sunk into the sand by means of a water jet, having a pressure of 65 lbs. per sq. in.)

COST OF MAKING AND PLACING REINFORCED CONCRETE PILES AT ATLANTIC CITY, N. J. *Engineering News*, v. 56, p. 252. (September 6, 1906.)

THE MANUFACTURE AND USE OF CONCRETE PILES. By Henry Longcope. *Proceedings National Association of Cement Users*, v. 2, p. 277. (1906.)

Abstracts of same. *Scientific American Supplement*, v. 61, p. 25375. (May 12, 1906.) *Municipal Engineering*, v. 30, p. 106. (February, 1906.)

REINFORCED CONCRETE PILE FOUNDATION FOR THE LATTEMANN BUILDING, BROOKLYN, N. Y. *Engineering News*, v. 54, p. 594. (December 7, 1905.) (The piles used were of the corrugated form invented by Frank B. Gilbreth.)

THE MAKING AND DRIVING OF CORRUGATED CONCRETE PILES. By Frank B. Gilbreth. Association of American Portland Cement Manufacturers, *Bulletin No. 7*. (1906.) (Illustrated.)

L'EMPLOI DES PIEUX EN BÉTON POUR LES FONDATIONS. *Le Genie Civil*, v. 49, p. 104. (June 16, 1906.) (An illustrated description of different types of concrete piles.)

Translation of same by George L. Fowler. THE USE OF CONCRETE PILES. *Railroad Gazette*, v. 41, p. 238. (September 21, 1906.)

APPONTEMENT MÉTALLIQUE DE LOME (AFRIQUE OCCIDENTALE). *Le Genie Civil*, v. 47, p. 178. (July 15, 1905.) (Contains a description of the concrete piles.)

CORRUGATED CONCRETE FOUNDATION PILES. *Engineering Record*, v. 52, p. 548. (November 11, 1905.) (Describes the method of constructing the foundations for the Lattemann Building. Brooklyn.)

BUILDING AND MACHINERY FOUNDATIONS IN QUICKSAND. *Engineering Record*, v. 53, p. 248. (March 3, 1906.) (For the Knickerbocker Building, New York City, the foundations of the column and wall piers consist of clusters of tubular steel piles 12 in. in diameter and ⅜ in. thick sunk to bed rock and filled with concrete.)

CONCRETE PILING. *Scientific American*, v. 90, p. 248. (March 26, 1904.)

DIE GRÜNDUNG DES AMTSGERICHTSGEBÄUDES AUF DEM WEDDING IN BERLIN MIT BETONEISENPFÄHLEN. By Hertel. *Beton und Eisen*, v. 2, p. 246. (October, 1903.)

REINFORCED CONCRETE PILING. By A. R. Galbraith. *Proceedings, Incorporated Association of Municipal and County Engineers*, v. 31, p. 356. (1904–5.) Spon & Chamberlain, 123 Liberty Street, New York.

Abstract of same. EUROPEAN REINFORCED CONCRETE PILES. *Engineering Record*, v. 52, p. 99. (July 22, 1905.)

CONCRETE PILES AT THE UNITED STATES NAVAL ACADEMY. By Walter R. Harper. *Engineering Record*, v. 51, p. 277. (March 4, 1905.) (Gives the comparative cost of wood and concrete piles; test of concrete pile and methods of construction.)

THE STRENGTH OF PILE AND CONCRETE FOUNDATIONS. *Engineering Record*, v. 50, p. 358. (September 24, 1904.) (Results of experiments made to determine the adhesion of timber piles to concrete, when imbedded in that material.)

CONCRETE PILE FOUNDATIONS AT WASHINGTON BARRACKS, D. C. By Captain John Stephen Sewell. *Engineering Record*, v. 50, p. 360. (September 24, 1904.) (2 pages.)

REINFORCED CONCRETE PILES WITH ENLARGED FOOTINGS FOR UNDERPINNING A BUILDING. By J. Albert Holmes. *Engineering Record*, v. 51, p. 567. (June 16, 1904.) (Describes the construction of piles for the foundation of a building in Boston, Mass.)

DES APPLICATIONS DU CIMENT ARMÉ. By G. Liébeaux, *Revue Generale des Chemins de Fer*, v. 24, Pt. 2, p. 525. (December, 1901.) (Contains illustrations of concrete piles for foundations.)

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CONCRETE-STEEL PILES. *Cement*, v. 4, p. 16. (March, 1903.) (A description of heavy pile drivers designed for driving concrete-steel piles.)

A TREATISE ON CONCRETE PLAIN AND REINFORCED. (P. 477.) By Frederick W. Taylor and Sanford E. Thompson, Associate Members, American Society Civil Engineers, N. Y., 1905. John Wiley & Sons, 43 East Nineteenth Street. $5.

CONCRETE AND REINFORCED CONCRETE CONSTRUCTION. (P. 428.) By Homer A. Reid, Associate Member, American Society Civil Engineers, N. Y. 1907. Myron C. Clark Publishing Company, 13 Park Row. $5 net. (Gives data on concrete piles.)

REINFORCED CONCRETE. (Pp. 41, 181, 200, 449.) By Charles F. Marsh, Member American Society Civil Engineers, and William Dunn. Ed. 3, London, 1906. Archibald Constable & Co., Ltd., 16 James Street, Haymarket. $7 net. (Very short references.)

CEMENT AND CONCRETE. (P. 485.) By Louis Carlton Sabin, Member American Society Civil Engineers. Ed. 2, New York, 1907. McGraw Publishing Company, 239 West Thirty-ninth Street. $5. (Contains 2 pages on concrete piles.)

CONCRETE STEEL. (P. 169.) By W. Noble Twelvetrees, New York, 1905. Whittaker & Co., 64 Fifth Avenue. $1.90. (Contains 10 pages on concrete-steel piles.)

REINFORCED CONCRETE PILES; THEIR MAKING AND DRIVING. *Cement*, v. 4, p. 331. (November, 1903.)

A NEW SYSTEM OF CONCRETE PILES. By W. P. Anderson, *Engineering Record*, v. 50, p. 494. (October 22, 1904.) (Piling for the Dittman factory building, Cincinnati, Ohio.)

CONCRETE PILES FOR BUILDING FOUNDATIONS. *Engineering Record*, v. 49, p. 596. (May 7, 1904.) (2 columns.)

A PRACTICAL TREATISE ON FOUNDATIONS. (Pp. 467, 477.) By W. M. Patton, Ed. 2, New York, 1906. John Wiley & Sons. $5. (Contains information on concrete piles.)

EINE NEUERE KAIMAUER MIT EISENBETON-PFAHLGRÜNDUNG. *Zeitschrift für Bauwesen*, v. 57, p. 550. (Pt. 10–12, 1907.) (On the reinforced concrete piles for the foundation of a new quay wall at Düsseldorf.)

METHODS AND COST OF DRIVING RAYMOND CONCRETE PILES FOR A BUILDING FOUNDATION. *Engineering-Contracting*. (February 13, 1907.) (Gives figures of cost computed from records obtained in constructing the pile foundations for a building in Salem, Mass.)

KÜNSTLICHE BEFESTIGUNG DES BAUBODENS MITTELST SCHWEBENDER PILOTAGE. By Ottokar Stern. *Beton und Eisen*. (January, 1907.) (Calculations and dimensions for concrete piling.)

CONCRETE PILES. By Charles R. Gow. *Journal Associated Engineering Society*. (October, 1907.) (Illustrated.)

THE SIMPLEX SYSTEM OF CONCRETE PILING. By Thomas MacKeller. *Journal Associated Engineering Society*. (October, 1907.)

PILOTIS "SIMPLEX." *Le Ciment*. (December, 1907.) (Illustrated.)

CONCRETE PILES—FORMS, ADVANTAGES AND COST AS COMPARED WITH WOODEN PILES. By C. W. Gaylord. *Proceedings*, v. 5, 1909. National Association of Cement Users. (21 pages. Illustrated.)

THE HANDLING OF CONCRETE IN THE CONSTRUCTION OF THE PANAMA CANAL.

By S. B. Williamson.*

The estimated amount of concrete that will be used in the construction of the Panama Canal aggregates 5,416,645 cu. yd. and is distributed as indicated in Table I.

Table I.—Status of Concrete Work on the Panama Canal on January 1, 1912.

Location.	Amounts in Cubic Yards.		
	Placed.	To be Placed.	Total.
Terminal Docks at Cristobal	500	51,800	52,300
Gatun Spillway Dam	167,500	57,580	225,080
Gatun Locks	1,761,345	238,895	2,000,240
Culebra Cut: Revetment	...	400,000	400,000
Pedro Miguel Locks	780,696	111,993	892,689
Miraflores Locks	571,860	840,876	1,412,736
Miraflores Spillway Dam	...	75,000	75,000
Terminal Docks at Balboa	6,535	330,065	336,600
Municipal Reservoirs, Dams, Power House, Bridges, etc	22,000	...	22,000
Total	3,310,436	2,106,209	5,416,645

While, no doubt, it is generally known that concrete plays a leading part in the building of the Canal, a summary of the quantities emphasizes its importance, which becomes still more significant when one realizes that the estimated cost of the concrete structures represents 23 per cent of the entire estimate for construction and engineering and that, had it been necessary to adopt stone masonry, the cost of these structures would have narrowly escaped a prohibitive figure, as there is no building stone within a reasonable distance from the Canal Zone. It does not seem out of the way, therefore, to advance the claim that the lock type of canal, now under construction, and acknowledged to be the most preferable by all engineers who have given the subject careful consideration, became feasible largely through the use of concrete.

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The locks comprise a large proportion of the total quantity of masonry and in order to place the amount of concrete involved, economically and at a rate to accomplish the work within the allotted time, it was necessary to devise especial and unusual handling appliances; and it is the purpose of the writer to describe the plants and methods adopted.

GATUN LOCKS.

General Description.—At Gatun the difference of 85 ft. between sea-level and the lake surface will be overcome by a flight of three locks, that is, the locks are directly connected, without intervening basins, and form a continuous structure. They are built in duplicate, which requires the construction of two side walls and a center wall, each 79 ft. high. The side walls are 4,038 ft. long; the center wall is extended in both directions to provide guide walls for vessels entering the locks and is 6,330 ft. in length. An outline plan and typical cross-sections of the Gatun flight of locks is shown on Fig. 1, Plate I, and 2,000,240 cu. yd. of concrete will be used in their construction.

The lock sites are about 3,300 ft. east of a channel that was dredged by the French Company from Colon to Gatun, and as the concrete aggregates were to be obtained from points on the coast and transported by water to Colon, it was considered advisable to continue this method of transportation to Gatun by utilizing the French Canal; the latter, therefore, at once became a controlling element in designing the handling plant. The banks of the French Canal are composed of an alluvial material, with a decided tendency to slide, and were considered unsafe for the heavy structures and storage piles required at the unloading point. A boat slip was therefore dredged in firmer ground between the canal and lock sites and a channel excavated to a connection with the French Canal—incidentally the movement of the unloading point shortened the distance between the storage piles and mixing plant.

Handling Plant.—Referring to Figs. 2 and 3, Plate II, it is seen that the entire plant is composed of the following units, each having a separate and distinct function:

1. Facilities for unloading and storing cement, sand and stone.

Berm Cranes.—Two berm cranes only were used at Pedro Miguel, but the booms were replaced by the cantilever arms of the other two so that, as erected, they were balanced cantilever cranes as shown in Fig. 13, Plate VII. Their runway tracks were so located between the trestles that each cantilever extended well over the respective storage piles; both cantilevers were equipped with trolleys operating a 2½-cu. yd. excavating bucket. The bins from the other cranes were also placed on these two, and with these and two cantilevers the facilities for handling and storing sand and stone were doubled. Aside from the above changes the previous

FIG. 12.—MIXING CRANES AND STORAGE TRESTLES, PEDRO MIGUEL LOCKS.

description of these cranes applies, except also that the swinging platforms were omitted and the mixers emptied into buckets on cars.

Narrow-Gauge Road.—The track system of the narrow-gauge road used for transporting concrete from the berm to chamber cranes is shown in Fig. 11. The difference of 30 ft. in elevation between the forebay and lock floors was overcome by means of an inclined trestle having a 2.5 per cent grade. The tracks were laid with 70-lb. rails which enabled the locomotives to attain a greater rate of speed than would be safe on the lighter rails usually employed. The equipment included twelve 11½-ton Porter

Plate VII.

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locomotives and 24 steel-framed flat cars, all equipped with air, each car being large enough to hold two 2-cu. yd. buckets.

The trains were composed of 2 cars and each car carried a bucket, so placed that when alongside a berm crane each bucket was filled from the corresponding mixer without moving the train. Usually the trains alternated in going into the respective lock chambers and stopped under the first chamber crane; the crane placed an empty bucket and picked up a loaded one from the same car; the train then moved to the next chamber crane where the operation of exchanging an empty bucket for a loaded one

FIG. 14.—GENERAL VIEW OF LOCKS AND CHAMBER CRANES, PEDRO MIGUEL LOCKS.

was repeated, after which the train of empty buckets returned to the mixers.

Chamber Cranes.—Two chamber cranes, Fig. 14, were erected in each lock at Pedro Miguel—they placed the concrete in both side and center walls. Except that the long cantilever arms extended over the side walls and the short ones over the center wall, the description of them given for the Miraflores plant applies.

Concrete Forms.—The same type of forms was used here and at Miraflores, in fact, all of the steel forming and some of the wood forms were used first at Pedro Miguel and later at Miraflores.

Auxiliary Mixing Plant.—Delays in the delivery and erection of the permanent plant, combined with the desire to increase the rate of placing concrete led to the erection of auxiliary mixing plants for the Pacific locks. At Pedro Miguel a 2-cu. yd. mixer was first temporarily set up at the lower end of the west wall and later moved to a similar position as regards the east wall, where it continued work until December 16, 1911. Two 2-cu. yd. cube mixers were installed under the south end of the storage trestle in the forebay and have continued in operation since. In all of the above cases the mixers were on hand, having been ordered for the permanent plant as applied to Miraflores where 8 mixers are required as against 4 in the application of the plant to Pedro Miguel. In each location the mixers were charged from bins to which material was delivered from tracks overhead, by standard railroad dump cars, and the product was transported to the walls by narrow-gauge equipment. Half-yard portable mixers are also used for floor construction and certain portions of the walls where they may be set up so as to pour concrete directly into the forms.

The stationary mixers located in the east wall at Miraflores have been previously described. As they are for the purpose of feeding the chamber cranes, and it was necessary to purchase additional mixers, they really constitute an addition to the permanent plant, made for the purpose of increasing its efficiency. Aside from these there is no auxiliary plant used at Miraflores, except the half-yard portable mixers.

PERFORMANCE OF PLANT.

For the Pacific locks, crushed stone is delivered into cars directly from the quarry bins and dumped from the cars on the storage trestles. Sand is dredged, loaded into barges and transferred from the latter to bins at Balboa with electric cranes, it then flows by gravity from the bins into cars for transporting to the storage trestles. There is nothing corresponding to the unloading plant at Gatun, therefore, unless it is the crane for handling sand at Balboa. Their performance is given in Table VII.

Pedro Miguel Locks.

Auxiliary Plant.—The placing of concrete in the lower guide wall at Pedro Miguel began on September 1, 1909, with a mixing

TABLE VII.—PERFORMANCE OF SAND UNLOADING CRANES, BALBOA SAND DOCK, PACIFIC DIVISION.

(Plant consists of three electrically operated cranes.)

FISCAL YEAR 1910–11.

Month.	Average Number of Cranes Working.	Sand Unloaded, cu. yd.	Service Time.							Rate per Hour per Crane, cu. yd.		Per cent of Working Time to Total Time under Pay.
			Hours Working.	Delay in Hours.					Total Hours per Month			
				Repairs to Cranes.	Waiting for Barges.	Waiting for Cars.	Other Delays.	Total Delays.		Working Time.	Total Time.	
July	2	36,299	282.36	9.80	30.64	73.92	3.28	117.64	400.00	128.56	90.75	70.59
August	2	40,277	301.28	41.84	57.14	15.34	0.83	115.15	416.43	133.69	96.72	72.33
September	2	42,500	253.42	10.00	90.75	30.50	15.33	146.58	400.00	167.70	106.25	63.34
October	2	26,432	268.80	9.00	105.20	32.00	1.00	147.20	416.00	172.73	111.62	64.62
November	2	43,161	311.50	13.50	31.00	28.50	0.50	73.50	385.00	138.56	112.11	80.94
December	2	42,520	297.00	10.50	57.00	51.50	5.00	124.00	421.00	143.16	01.00	70.37
January	2	41,130	320.50	12.50	......	70.00	.....	82.50	403.00	128.33	102.06	79.54
February	2	36,272	254.00	2.00	......	112.00	.....	114.00	368.00	142.80	98.57	69.00
March	2	41,878	300.00	......	......	132.00	.....	132.00	432.00	139.59	96.94	69.44
April	2	36,663	271.00	......	......	113.00	.....	113.00	384.00	135.29	95.48	71.09
May	2	35,903	286.00	2.00	......	114.00	14.00	130.00	416.00	125.53	86.31	68.75
June	2	51,806	283.00	8.00	......	125.00	.....	133.00	416.00	183.06	124.53	68.02
Total	2	494,841	3,428.86	119.14	371.73	897.76	39.94	1,428.56	4,857.43	144.32	101.87	70.59
Per cent of total	.	......	70.59	2.45	7.66	18.48	.82	29.41	100.00	......	......	.....

FIRST SEVEN MONTHS OF FISCAL YEAR 1911–1912.

Month.	Average Number of Cranes Working.	Sand Unloaded, cu. yd.	Hours Working.	Repairs to Cranes.	Waiting for Barges.	Waiting for Cars.	Other Delays.	Total Delays.	Total Hours per Month	Working Time.	Total Time.	Per cent of Working Time to Total Time under Pay.
July	2	34,000	255.00	......	......	143.00	2.00	145.00	400.00	133.33	85.00	63.75
August	2	40,900	284.00	1.00	......	142.00	5.00	148.00	432.00	144.01	94.68	65.74
September	1.96	38,100	260.50	8.50	......	123.00	.....	131.50	392.00	146.25	97.19	66.74
October	2	46,000	279.00	......	......	145.00	8.00	153.00	432.00	164.87	106.48	64.58
November	2	34,700	234.00	......	......	150.00	.....	150.00	384.00	148.09	93.64	69.37
December	2	40,700	250.00	......	......	150.00	.....	150.00	400.00	162.80	101.75	62.50
January	2	48,550	304.00	......	4.00	103.00	1.00	108.00	412.00	159.70	117.84	62.50
Total	1.99	283,850	1,866.50	9.50	4.00	956.00	16.00	985.50	2,852.00	152.08	103.03	65.45

TABLE VIII.—PERFORMANCE OF AUXILIARY CONCRETE-HANDLING PLANT, PEDRO MIGUEL, PACIFIC DIVISION.

PART 1.—FISCAL YEAR 1909–1910.

Month.	Mixers Used.		Working Time (Average per Mixer).			Concrete Placed (all Mixers), cu. yd.			
	Average Number.	Size, cu. yd.	Days of 8 Hours.	Hours. Per Day of 8 Hours.	Hours. Total per Day of 8 Hours.	Per Day of 8 Hours.	Per Hour.	Total.	Per Mixer Hour.
July	....	..	..	....	......	......	......	....	.....
August	....	...	..	....	......	......	......	....	.....
September	1.00	2	5	5.48	27.42	92.40	16.85	462.00	16.85
	1.00	½	21	7.31	153.58	75.54	10.33	1,586.50	10.33
October	1.00	2	25	6.13	153.25	136.80	22.32	3,420.00	22.32
	1.46	½	29	8.12	227.97	133.74	16.47	3,755.50	11.28
November	1.00	2	23	6.14	141.17	152.26	24.80	3,502.00	24.80
	3.17	½	23	7.90	181.70	262.54	33.22	6,046.50	10.48
December	1.12	2	26	6.13	159.38	164.53	26.84	4,260.00	23.96
	3.73	½	26	8.45	219.70	343.46	40.66	8,932.00	10.90
January	1.88	2	25	6.80	162.00	414.41	63.96	10,360.00	34.02
	3.12	½	25	8.01	200.25	277.80	34.66	6,945.50	11.11
February	1.91	2	23	7.15	164.45	347.81	48.65	8,012.00	25.47
	2.22	½	23	8.38	192.74	223.04	26.62	5,124.00	11.99
March	1.46	2	28	7.26	203.28	274.73	37.86	7,715.00	25.93
	1.93	½	27	8.23	222.21	200.00	24.30	5,388.50	12.59
April	1.10	2	10	5.95	59.50	93.80	15.75	938.00	14.32
	1.69	½	26	7.75	201.50	132.63	17.10	3,453.00	10.12
May	1.00	2	23	6.38	146.83	170.80	26.77	3,928.00	26.77
	1.82	½	23	6.74	155.02	170.20	15.91	2,474.00	8.74
June	1.00	2	27	7.87	212.59	244.78	31.08	6,609.00	31 08

PART 2.—FISCAL YEAR 1910–1911.

Month.	Average Number.	Size, cu. yd.	Days of 8 Hours.	Hours. Per Day of 8 Hours.	Hours. Total per Day of 8 Hours.	Per Day of 8 Hours.	Per Hour.	Total.	Per Mixer Hour.
July	1.00	2	25	6.81	170.33	203.60	29.88	5,090.00	29.88
August	1.00	2	1	9.00	9.00	154.00	17.11	154.00	17.11
September	1.00	2	9	4.83	43.50	99.56	20.60	896.00	20.60
October	1.42	2	24	3.75	89.88	230.50	61.54	5,532.00	43.34
November	2.00	2	24	5.77	138.45	510.42	88.48	12,250.00	44.24
December	2.42	2	26	4.53	117.75	451.65	99.73	11,743.00	41.21
January	1.67	2	24	5.00	130.11	299.84	59.92	7,196.00	35.88
February	2.61	2	23	5.18	119.10	550.96	106.41	12,672.00	40.77
	1.00	½	17	2.16	36.75	22.35	10.34	380.00	10.34
March	2.93	2	27	5.18	139.99	639.00	123.24	17,253.00	42.06
	1.00	½	24	7.10	170.50	115.71	16.29	2,777.00	16.29
April	2.00	2	24	5.39	129.38	455.75	84.54	10,938.00	42.27
	1.57	½	23	5.45	125.32	122.56	22.50	2,819.00	14.33
May	1.65	2	26	5.45	141.07	476.54	87.83	12,390.00	53.23
	1.32	½	22	5.00	110.04	77.55	15.51	1,706.00	11.75
June	2.78	2	27	5.18	139.93	626.04	120.79	16,903.00	43.45
	1.29	½	17	5.48	93.16	96.82	17.67	1,646.00	13.70

PART 3.—FIRST SEVEN MONTHS OF YEAR 1912.

Month.	Average Number of Mixers.	Concrete Mixed, cu. yd.	Hours Mixing.	Delays, in Hours.						Total Unit Hours per Day	Cubic Yards Mixed per Hour.		Per cent of Hours at Work to Hours Operated.
				Repairs.	Waiting for Material.	Waiting for Forms.	Waiting for Cars.	Other Delays.	Total Delays.		Working Time.	Total Time.	
July	2.96	16,333	420.00	21.00	5.50	39.00	168.50	12.00	246.00	666.00	38.9	24.5	63.06
August	3.00	15,896	445.50	8.50	13.00	57.50	202.50	2.00	283.50	729.00	35.7	21.8	61.11
September	2.76	13,088	367.00	.....	10.50	106.50	133.00	4.00	254.00	621.00	35.7	21.1	59.10
October	2.88	16,371	446.00	3.50	13.50	141.00	38.50	5.50	202.00	648.00	36.7	25.3	68.83
November	3.00	14,220	374.00	7.50	16.00	198.00	39.00	13.50	274.00	648.00	38.00	21.9	57.72
December	2.40	10,494	318.00	.....	24.00	168.00	15.00	15.00	222.00	540.00	33.0	19.4	58.89
January	2.00	10,122	306.42	1.50	55.58	96.50	......	8.00	161.58	468.00	33.3	21.6	65.47
Total	2.70	96,524	2,676.92	42.00	138.08	806.50	595.50	60.00	1,643.08	4,320.00	36.06	22.34	61.97

plant consisting of two 2-cu. yd. and three ½-cu. yd. mixers. The performance of this equipment to June 30, 1910, working on an 8-hr. day basis, is detailed in Part 1 of Table VIII.

During the fiscal year ending June 30, 1911, the auxiliary plant consisted mainly of three 2-cu. yd. mixers, one located at the south end of the east wall and two in the forebay, the half-yard having been transferred to Miraflores. It placed 121,530 cu. yd. and the detailed performance, on an 8-hr. day basis, is shown in Part 2 of Table VIII; and the detailed performance, on an 8-hr. day basis, of this plant for the first seven months of the fiscal year ending June 30, 1912, is shown in Part 3 of the same table.

Permanent Plant.—One berm and two chamber cranes, or one-half of the plant, began operating on April 4, 1910 (a chamber crane placed some concrete from auxiliary mixers in March); the other half began on July 15, 1910. The portion of the permanent plant in operation laid 73,083 cu. yd. prior to June 30, 1910, on an 8-hr. day basis, as detailed in Table IX.

The plant worked as a whole from July 15, 1910, to January 31, 1911. On the latter date the dismantling of one berm crane began preparatory to erecting it at Miraflores. The dismantling of two of the chamber cranes for the same purpose began on April 20 and May 9, 1911, respectively, and that of the remaining berm crane on May 19, 1911. Two chamber cranes remained at Pedro Miguel until December 12, 1911, and January 31, 1912, respectively, being used in the meantime for placing concrete from auxiliary mixers, setting iron work and backfilling the middle wall. The plant placed 379,190 cubic yards during the fiscal year 1911, and the detailed performances of the berm and chamber cranes respectively are given in Tables X and XI.

Miraflores Locks.

Auxiliary Plant.—Placing concrete in the floors and lateral culverts of the upper locks at Miraflores was begun on June 1, 1910, with a plant consisting of two ½-cu. yd. mixers, and 1630 cu. yd. were placed before June 30, 1910.

In the fiscal year ending June 30, 1911, the auxiliary plant

TABLE IX.—PERFORMANCE OF PERMANENT CONCRETE CONSTRUCTION PLANT AT PEDRO MIGUEL, PACIFIC DIVISION.

LAST SIX MONTHS OF FISCAL YEAR 1909–1910.

Month.	Mixers Used.		Working Time (Average per Mixer).			Concrete Placed (all Mixers), cu. yd.			
				Hours.					
	Average Number.	Size, cu. yd.	Days of 8 Hours	Per Day of 8 Hours.	Total per Day of 8 Hours.	Per Day of 8 Hours.	Per Hour.	Total.	Per Mixer Hour.
January	....	.	..	....	......	......	......	......	.....
February	....	.	..	....	......	......	......	......	.....
March	1.00	2	14	6.11	85.50	385.54	63.10	5,404	63.10
April	1.88	2	26	7.47	194.67	791.07	105.81	20,615	56.28
May	2.00	2	25	7.38	184.58	926.63	125.56	23,178	62.78
June	2.52	2	27	6.15	166.05	885.42	143.97	23,886	57.13

TABLE X.—PERFORMANCE OF BERM CRANES, PEDRO MIGUEL LOCK, PACIFIC DIVISION.

FIRST ELEVEN MONTHS OF FISCAL YEAR 1910–1911.

Month	Number of Cranes.	Number of Mixers. (2 cu. yd.)	Concrete Mixed, cu. yd.	Service Time of Mixers.							Rate per Hour per Mixer, cu. yd.		Per cent of Working Time to Total Time under Pay.
					Delay, in Hours.								
				Hours Working.	Repairs to Cranes.	Repairs to Mixers.	Waiting for Cars.	Other Delays.	Total Delays.	Total Hours per Month.	Working Time.	Total Time.	
July	2.00	4.00	34,890	630.50	49.50	1.67	214.16	4.17	269.50	900.00	55.33	38.76	70.16
August	2.00	4.00	50,808	767.37	34.21	4.33	152.67	13.42	204.63	972.00	66.21	52.27	79.85
September	2.00	4.00	49,852	683.09	15.30	24.08	170.00	10.53	219.91	903.00	72.98	55.21	75.62
October	1.96	3.93	55,152	686.33	96.35	12.67	157.07	1.00	267.09	953.42	80.71	58.96	72.14
November	2.00	4.00	49,568	622.82	45.75	3.50	181.27	8.83	239.35	862.17	79.59	57.49	72.24
December	1.73	3.15	31,156	427.49	8.34	.....	275.52	2.18	286.04	713.53	72.88	43.66	59.94
January	1.72	2.76	35,640	427.42	12.66	5.00	131.99	6.84	156.49	583.91	83.38	61.04	73.14
February	1.00	2.00	23,770	321.33	25.00	2.32	62.68	6.67	96.67	418.00	73.97	56.87	76.87
March	1.00	2.00	25,704	396.51	13.67	1.50	103.16	11.16	129.49	526.00	64.83	48.87	75.28
April	1.00	2.00	17,188	296.33	22.00	.67	105.50	7.50	135.67	432.00	58.00	39.79	68.59
May	1.00	2.00	5,462	135.00	11.00	.....	160.00	.....	171.00	306.00	40.46	17.84	44.12
Total	1.58	3.08	379,190	5,394.19	333.78	55.74	1,714.02	72.30	2,175.84	7,570.03	70.30	50.09	71.26
Per cent of total	....	....	......	72.40	4.44	.77	21.39	1.00	——	100.00	.....	.....	.....

Dismantling of second crane began on May 19, 1911, preparatory to removal to Miraflores Locks.

TABLE XI.—PERFORMANCE OF CHAMBER CRANES, PEDRO MIGUEL LOCK, PACIFIC DIVISION.

(Plant consists of four electrically operated cranes.)

FISCAL YEAR 1910–1911.

Month.	Average Number of Cranes.	Concrete Placed.	Hours Working.			Delays, in Hours.						Total Unit Hours Operated per Month.	Concrete Handled per Hour, cu. yd.		Per cent of Hours at Work to Hours Operated.	Filling Placed in Center Wall, cu. yd.
			Placing Concrete.	Handling Steel, Forms, Filling, etc.	Total.	Repairs to Cranes.	Waiting for Concrete.	Waiting for Forms.	Bridging.*	Other Delays.	Total Delays.		Working Time.	Total Time.		
July	3.56	32,326	639.30	....	639.30	31.32	41.53	16.59	.....	17.28	106.72	746.02	50.56	43.33	85.57	....
August	4.00	48,558	786.51	....	786.51	55.97	43.27	19.25	.....	24.98	143.47	929.98	61.74	52.21	84.60	....
September	3.96	49,488	659.41	....	659.41	23.69	78.67	60.76	.....	39.82	202.94	862.35	75.05	57.39	76.64	....
October	3.93	54,263	710.46	....	710.46	28.92	138.82	38.85	.....	40.99	247.58	958.04	76.38	56.64	74.15	....
November	4.00	48,453	647.97	....	647.97	24.67	70.87	85.99	.....	19.67	201.20	849.17	74.78	57.06	76.34	....
December	3.66	31,047	466.09	....	466.09	18.04	25.67	269.76	.....	46.47	359.94	826.03	66.61	37.58	56.46	....
January	3.68	33,902	524.80	....	524.80	14.15	49.77	168.09	.....	43.22	275.23	800.03	64.59	42.38	65.63	....
February	4.00	28,426	483.19	....	483.19	15.21	67.43	185.90	.....	66.27	334.81	818.00	58.83	34.75	59.07	....
March	4.00	35,118	588.95	....	588.95	9.63	129.41	157.34	.....	118.67	415.05	1,004.00	59.63	34.98	58.66	....
April	3.40	22,602	390.31	....	390.31	23.76	73.23	166.68	.....	84.02	347.69	738.00	57.91	30.63	52.88	....
May	1.79	11,388	199.70	....	199.70	4.76	69.37	52.95	.....	60.22	187.30	387.00	57.03	29.43	52.83	....
June	1.38	7,584	133.33	....	133.33	2.08	75.42	53.01	.....	55.66	186.17	319.50	56.88	23.74	41.73	....
Total	3.45	403,555	6,230.02	....	6,230.02	252.20	863.46	1,275.17	.....	617.27	3,008.10	9,238.10	64.71	43.64	67.44	....
Percent of total	....	......	67.44	....	67.44	2.73	9.35	13.80	.....	6.68	32.56	100.00	.....	.....	.....	....

* Time shown in "Other Delays" column.

FIRST SEVEN MONTHS OF FISCAL YEAR 1911–1912.

Month.	Average Number of Cranes.	Concrete Placed.	Placing Concrete.	Handling Steel, Forms, Filling, etc.	Total.	Repairs to Cranes.	Waiting for Concrete.	Waiting for Forms.	Bridging.*	Other Delays.	Total Delays.	Total Unit Hours Operated per Month.	Working Time.	Total Time.	Per cent of Hours at Work to Hours Operated.	Filling Placed in Center Wall, cu. yd.
July	1.96	7,126	133.97	106.59	240.56	14.72	69.02	87.29	18.06	11.35	200.44	441.00	53.2	16.2	54.55	....
August	1.89	4,660	103.58	160.32	263.90	7.84	36.94	107.42	12.35	30.55	195.10	459.00	44.9	10.2	57.28	....
September	2.00	2,374	69.26	257.33	326.59	6.09	28.83	70.50	10.99	7.00	123.41	450.00	34.3	5.3	72.58	....
October	2.00	6,459	153.70	228.34	382.04	29.50	28.23	38.00	28.73	15.50	139.96	522.00	42.0	12.4	73.19	....
November	2.00	6,009	139.34	191.38	330.72	7.51	61.16	16.10	16.51	......	101.28	432.00	43.2	13.9	77.02	....
December	1.48	1,116	52.58	110.83	163.41	9.42	31.75	46.67	17.34	64.41	169.59	333.00	17.5	7.4	49.07	1,362
January	1.48	706	17.25	93.49	110.74	12.25	5.58	1.08	39.41	137.68	196.00	306.74	17.4	6.3	36.10	....
Total	1.89	28,450	669.68	1,148.28	1,817.96	87.33	261.51	367.06	143.39	266.49	1,125.78	2,943.74	42.48	9.66	61.76	....

Dismantling last crane for removal to Miraflores commenced Monday, January 29, 1912.

consisted of two 2-cu. yd. mixers located under the north end of the east storage trestle, and four ½-cu. yd. mixers. The 2-cu. yd. mixer plant was moved to the east wall for supplying the chambers cranes in May, 1911: previous to this it supplied a berm crane that was sufficiently complete to place concrete with the boom, though its mixers and cantilever arm were still in use at Pedro Miguel. The auxiliary plant handled 205,255 cu. yd. during the year, as detailed in Table XII; and a similar statement giving the detailed performance of the 2-cu. yd. auxiliary mixers at Miraflores, on an 8-hr. day basis, for the first seven months of the fiscal year ending June 30, 1912, appears in Table XIII.

Permanent Plant.—One berm crane without cantilever arm and mixers began, in its uncompleted condition, to place concrete supplied by the auxiliary mixers on September 2, 1910, and continued placing until February 15, 1911, when it was taken out of commission for completion. It began operating again, as a complete machine, on March 22, 1911, and a second berm crane began work on April 7, 1911. With these 67,774 cu. yd. were laid previous to June 30, 1911.

Other units of the plant were placed in commission as follows: 2 chamber cranes on July 13 and August 3, 1911, respectively, and 2 additional berm cranes on July 25 and October 28, 1911, respectively. There are 2 more chamber cranes under erection as follows: Table XIV shows the performance of berm cranes, Miraflores Locks, to January 31, 1912, and Table XV the performance of chamber cranes, Miraflores Locks, for first seven months of fiscal year ending June 30, 1912.

The performances of the Pacific Division plants for the fiscal year 1911 are given by the Cost Accountant as follows: In Pedro Miguel locks 497,802 cu. yd. of concrete were placed, average division cost $4.70 per cu. yd. and 385 cu. yd. of reinforced concrete at $17.74 or a total of 498,187 cu. yd. at $4.71 per cu. yd. In Miraflores locks 272,933 cu. yd. were laid at an average division cost of $4.68 per cu. yd. The lowest average cost for any one month was for Pedro Miguel in November, 1910, when 64,248 cu. yd. were placed at $4.20 a cu. yd. and for Miraflores in May, 1911, when 36,154 cu. yd. were placed at $4.05 per cu. yd.

TABLE XII.—AUXILIARY PLANT, MIRAFLORES LOCKS: PERFORMANCE OF MIXERS, PACIFIC DIVISION.

FISCAL YEAR 1910–1911.

Month.	Mixers Used.		Working Time (Average per Mixer).			Concrete Placed (all Mixers), cu. yd.			
	Average Number.	Size, cu. yd.	Days of 8 Hours.	Hours. Per Day of 8 Hours.	Hours. Total per Day of 8 Hours.	Per Day of 8 Hours.	Per Hour.	Total.	Per Mixer Hour.
July	1.73	½	26	7.81	203.13	117.46	8.69	3,054	5.02
	1.00	⅓	22	7.38	162.33	28.11	3.81	618	3.81
August	2.00	½	27	7.64	206.17	171.14	11.21	4,621	5.60
	1.00	⅓	27	7.83	211.41	40.78	5.21	1,101	5.21
September	1.46	2	24	5.60	134.50	382.83	46.79	9,188	32.05
	3.56	½	25	6.95	173.88	316.48	12.78	7,912	3.59
	1.00	⅓	15	6.36	95.30	46.33	7.29	695	7.29
October	2.00	2	27	3.57	96.50	373.93	52.31	10,096	26.16
	3.43	½	28	6.91	193.51	409.48	17.27	11,465	5.04
November	2.00	2	24	4.22	101.21	482.67	57.23	11,584	28.61
	3.58	½	24	6.73	161.47	405.35	16.83	9,729	4.70
	1.00	⅓	24	7.10	170.41	78.40	11.04	1,881	11.04
December	2.00	2	26	3.88	100.79	453.77	58.52	11,798	29.26
	3.27	½	26	6.05	157.31	294.92	14.91	7,668	4.56
	1.00	⅓	24	5.76	138.25	67.46	11.71	1,619	11.71
January	2.00	2	25	5.80	144.95	457.76	39.48	11,444	19.74
	3.92	½	25	6.43	160.73	398.16	15.80	9,954	4.03
	1.00	⅓	23	6.55	150.70	84.87	12.95	1,952	12.95
February	1.87	2	23	4.72	108.55	424.35	48.08	9,760	25.71
	3.70	½	23	6.15	141.35	384.26	16.90	8,838	4.57
	1.00	⅓	21	6.37	133.75	84.00	13.19	1,764	13.19
March	2.00	2	27	3.29	88.90	295.74	44.90	7,984	22.45
	5.26	½	27	7.20	194.31	699.11	18.49	18,876	3.51
	1.00	⅓	26	5.79	150.50	70.46	12.17	1,832	12.17
April	2.00	2	24	2.40	57.59	242.75	50.62	5,826	25.31
	6.08	½	24	7.11	170.59	720.54	16.67	17,293	2.74
	1.00	⅓	16	7.20	115.19	70.06	9.73	1,121	9.73
May	1.92	2	22	3.24	71.27	170.73	27.59	3,756	14.45
	3.28	½	25	5.97	149.24	349.84	17.87	8,746	5.45
	1.00	⅓	17	6.06	103.00	57.18	9.44	972	9.44
June	3.19	½	26	6.00	155.88	301.15	15.75	7,830	4.94

TABLE XIII.—PERFORMANCE OF 2-CU. YD. AUXILIARY MIXERS, MIRAFLORES LOCKS, PACIFIC DIVISION.

FIRST SEVEN MONTHS OF FISCAL YEAR ENDING JUNE 30, 1912.

Month.	Average Number of Mixers Used (2 yd.)	Concrete Mixed, cu. yd.	Hours Mixing.	Delays, in Hours.						Total Unit Hours per Month	Cubic Yards Mixed per Hour.		Per cent of Hours at Work to Hours Operated.
				Repairs.	Waiting for Material.	Waiting for Forms.	Waiting for Cars.	Other Delays.	Total Delays.		Working Time.	Total Time.	
July	1.94	7,768	180.42	3.25	.....	.....	73.58	21.75	98.58	279.00	43.1	27.8	64.67
August	2.00	23,512	373.80	7.30	4.83	.....	73.39	32.68	118.20	492.00	62.9	47.8	75.98
September	2.00	27,874	379.73	4.77	8.05	3.00	31.92	41.53	89.27	469.00	73.4	59.4	80.97
October	2.00	26,406	378.01	8.70	6.40	13.70	29.03	57.66	115.49	493.50	69.9	53.5	76.59
November	1.83	15,536	228.60	17.00	6.80	43.60	89.70	34.30	191.40	420.00	67.9	37.0	54.43
December	1.92	12,322	190.30	32.94	.94	.....	......	234.82	268.70	459.00	64.7	26.9	41.46
January	1.92	6,360	95.62	9.00	3.50	.....	......	305.88	318.38	414.00	66.5	15.4	23.10
Total	1.91	119,778	1,826.48	82.96	30.52	60.30	297.62	728.62	1,200.02	3,026.50	65.58	39.58	60.42

TABLE XIV.—PERFORMANCE OF BERM CRANES, MIRAFLORES LOCKS, PACIFIC DIVISION.

Month.	Average Number of Cranes	Average Number of Mixers 2 cu. yd.	Concrete Mixed, cu. yd.	Hours Working. Mixing and Placing.	Hours Working. Handling Steel, Forms, and Other Material.	Hours Working. Total.	Delays, in Hours. Repairs to Cranes	Delays, in Hours. Repairs to Mixers	Delays, in Hours. Waiting for Material	Delays, in Hours. Waiting for Cars.	Delays, in Hours. Waiting for Forms.	Delays, in Hours. Bridging.	Delays, in Hours. Other Delays	Delays, in Hours. Total Delays	Total Crane Hours.	Mixer Hours. Working.	Mixer Hours. Total.	Rate per Hour per Mixer, cu. yd. Working Time.	Rate per Hour per Mixer, cu. yd. Total Time.	Per cent of Working Time to Total Time Under Pay.
LAST FOUR MONTHS OF FISCAL YEAR 1910–1911.																				
March....	1.00	2.00	2,122	67.84	......	67.84	8.00	.....	.....	.....	43.50	......	42.66	94.16	162.00	....	....	31.28	13.10	41.88
April......	1.82	3.65	16,736	469.42	......	469.42	58.75	10.25	18.83	.....	52.84	......	127.91	268.58	738.00	....	....	35.65	22.68	63.61
May......	2.00	3.84	24,480	564.73	......	564.73	54.66	5.99	.....	2.50	....	......	227.12	290.27	855.00	....	....	43.35	28.63	66.05
June......	2.00	3.84	24,436	550.80	......	550.80	70.41	11.68	29.68	.....	67.28	......	170.15	349.20	900.00	....	....	44.36	27.15	61.20
Total.....	1.80	3.59	67,774	1,652.79	......	1,652.79	191.82	27.92	48.51	2.50	163.62	......	567.84	1,002.21	2,655.00	....	....	41.69	25.53	62.25
Per cent of total....	....	....	......	62.25	......	62.25	7.23	1.05	1.83	.09	6.16	......	21.39	37.75	100.00	....	....	.....	.....	.....
FIRST SEVEN MONTHS OF FISCAL YEAR 1911–1912.																				
July......	2.24	4.20	25,326	273.45	46.87	320.32	53.58	5.50	.....	.....	61.17	33.43	30.00	183.68	504.00	512.68	945.00	49.40	26.80	63.55
August....	3.00	5.26	33,726	387.21	49.28	436.49	67.90	4.00	.....	.....	97.65	67.36	55.60	292.51	729.00	678.85	1,278.00	49.70	26.40	59.87
September.	2.88	5.28	25,349	309.07	22.68	331.75	44.94	3.00	.....	.....	158.84	62.87	46.60	316.25	648.00	566.70	1,188.00	44.70	21.30	51.20
October...	3.04	5.85	26,200	367.09	27.92	395.01	45.64	1.00	.....	.....	148.34	75.17	45.84	315.99	711.00	706.28	1,368.00	30.70	19.10	55.56
November.	4.00	7.00	19,427	373.50	61.00	434.50	45.25	2.50	.....	.....	262.75	78.55	40.45	429.50	864.00	653.62	1,572.00	29.70	12.80	50.17
December.	4.00	6.64	31,180	560.15	72.70	632.85	47.69	6.92	.....	.....	136.32	62.19	63.03	316.15	949.00	929.85	1,575.34	33.50	19.80	66.69
January...	4.00	6.31	37,054	592.76	119.98	712.74	42.00	4.18	.....	.....	147.36	73.26	83.46	350.26	1,063.00	971.54	1,742.25	38.10	21.30	67.05
Total.....	3.30	5.78	198,262	2,863.23	400.43	3,263.66	347.00	27.10	.....	...	1,012.43	452.83	364.98	2,204.34	5,468.00	5,019.52	9,608.59	39.50	20.63	67.05

TABLE XV.—PERFORMANCE OF CHAMBER CRANES FOR MIRAFLORES LOCKS, PACIFIC DIVISION.

FISCAL YEAR ENDED JUNE 30, 1912.

Month.	Average Number of Cranes.	Concrete Placed, cu. yd.	Hours Working.			Delays, in Hours.						Total Unit Hours Operated per Month.	Concrete Handled per Hour, cu. yd.		Per cent of Work Hours at to Hours Operated.
			Placing Concrete.	Handling Steel, Forms, Filling, etc.	Total.	Repairs to Cranes.	Waiting for Concrete.	Waiting for Forms.	Bridging.	Other Delays.	Total Delays.		Working Time.	Total Time.	
July..........	1.00	7,722	122.90	.50	123.40	4.44	11.99	1.34	1.83	1.00	20.60	144.00	62.80	53.60	85.69
August........	1.93	23,460	404.78	6.82	411.60	28.45	17.78	6.75	9.42	.....	62.40	474.00	57.90	49.50	86.83
September.....	2.00	27,536	413.69	1.87	415.56	14.93	34.83	......	3.68	.....	53.44	469.00	66.60	58.70	88.60
October.......	2.00	24,486	419.85	6.75	426.60	5.67	40.98	16.75	1.50	2.00	66.90	493.50	58.30	55.70	86.44
November.....	1.92	14,782	293.91	24.46	318.37	11.68	32.83	65.72	8.40	.....	118.63	437.00	50.30	33.80	72.81
December.....	1.96	10,698	226.72	45.17	271.89	4.34	40.96	107.38	22.49	10.27	185.44	457.33	47.20	23.40	59.45
January.......	1.96	4,244	105.74	160.90	266.64	.92	18.59	94.18	63.09	15.58	192.36	459.00	40.10	9.00	58.09
Total.........	1.87	112,928	1,987.59	246.47	2,234.06	70.43	197.96	292.12	110.41	28.85	699.77	2,933.83	56.82	38.49	76.15

First crane commenced operation July 13th.

TABLE XIV.—PERFORMANCE OF BERM CRANES, MIRAFLORES LOCKS, PACIFIC DIVISION.

Month.	Average Number of Cranes	Average Number of Mixers 2 cu. yd.	Concrete Mixed, cu. yd.	Hours Working. Mixing and Placing.	Hours Working. Handling Steel, Forms, and Other Material.	Hours Working. Total.	Delays, in Hours. Repairs to Cranes	Delays, in Hours. Repairs to Mixers	Delays, in Hours. Waiting for Material	Delays, in Hours. Waiting for Cars.	Delays, in Hours. Waiting for Forms.	Delays, in Hours. Bridging.	Delays, in Hours. Other Delays	Delays, in Hours. Total Delays	Total Crane Hours.	Mixer Hours. Working.	Mixer Hours. Total.	Rate per Hour per Mixer, cu. yd. Working Time.	Rate per Hour per Mixer, cu. yd. Total Time.	Per cent of Working Time to Total Time Under Pay.
LAST FOUR MONTHS OF FISCAL YEAR 1910–1911.																				
March....	1.00	2.00	2,122	67.84	......	67.84	8.00	.....	.....	.....	43.50	......	42.66	94.16	162.00	....	....	31.28	13.10	41.88
April......	1.82	3.65	16,736	469.42	......	469.42	58.75	10.25	18.83	.....	52.84	......	127.91	268.58	738.00	....	....	35.65	22.68	63.61
May......	2.00	3.84	24,480	564.73	......	564.73	54.66	5.99	.....	2.50	....	......	227.12	290.27	855.00	....	....	43.35	28.63	66.05
June......	2.00	3.84	24,436	550.80	......	550.80	70.41	11.68	29.68	.....	67.28	......	170.15	349.20	900.00	....	....	44.36	27.15	61.20
Total.....	1.80	3.59	67,774	1,652.79	......	1,652.79	191.82	27.92	48.51	2.50	163.62	......	567.84	1,002.21	2,655.00	....	....	41.69	25.53	62.25
Per cent of total....	....	....	......	62.25	......	62.25	7.23	1.05	1.83	.09	6.16	......	21.39	37.75	100.00	....	....	......	......	.....
FIRST SEVEN MONTHS OF FISCAL YEAR 1911–1912.																				
July......	2.24	4.20	25,326	273.45	46.87	320.32	53.58	5.50	.....	.....	61.17	33.43	30.00	183.68	504.00	512.68	945.00	49.40	26.80	63.55
August....	3.00	5.26	33,726	387.21	49.28	436.49	67.90	4.00	.....	.....	97.65	67.36	55.60	292.51	729.00	678.85	1,278.00	49.70	26.40	59.87
September.	2.88	5.28	25,349	309.07	22.68	331.75	44.94	3.00	.....	.....	158.84	62.87	46.60	316.25	648.00	566.70	1,188.00	44.70	21.30	51.20
October...	3.04	5.85	26,200	367.09	27.92	395.01	45.64	1.00	.....	.....	148.34	75.17	45.84	315.99	711.00	706.28	1,368.00	30.70	19.10	55.56
November.	4.00	7.00	19,427	373.50	61.00	434.50	45.25	2.50	.....	.....	262.75	78.55	40.45	429.50	864.00	653.62	1,572.00	29.70	12.80	50.17
December.	4.00	6.64	31,180	560.15	72.70	632.85	47.69	6.92	.....	.....	136.32	62.19	63.03	316.15	949.00	929.85	1,575.34	33.50	19.80	66.69
January...	4.00	6.31	37,054	592.76	119.98	712.74	42.00	4.18	.....	.....	147.36	73.26	83.46	350.26	1,063.00	971.54	1,742.25	38.10	21.30	67.05
Total.....	3.30	5.78	198,262	2,863.23	400.43	3,263.66	347.00	27.10	.....	...	1,012.43	452.83	364.98	2,204.34	5,468.00	5,019.52	9,608.59	39.50	20.63	67.05

TABLE XV.—PERFORMANCE OF CHAMBER CRANES FOR MIRAFLORES LOCKS, PACIFIC DIVISION.

FISCAL YEAR ENDED JUNE 30, 1912.

Month.	Average Number of Cranes.	Concrete Placed, cu. yd.	Hours Working.			Delays, in Hours.						Total Unit Hours Operated per Month.	Concrete Handled per Hour, cu. yd.		Per cent of Work Hours at to Hours Operated.
			Placing Concrete.	Handling Steel, Forms, Filling, etc.	Total.	Repairs to Cranes.	Waiting for Concrete.	Waiting for Forms.	Bridging.	Other Delays.	Total Delays.		Working Time.	Total Time.	
July..........	1.00	7,722	122.90	.50	123.40	4.44	11.99	1.34	1.83	1.00	20.60	144.00	62.80	53.60	85.69
August........	1.93	23,460	404.78	6.82	411.60	28.45	17.78	6.75	9.42	.....	62.40	474.00	57.90	49.50	86.83
September.....	2.00	27,536	413.69	1.87	415.56	14.93	34.83	......	3.68	.....	53.44	469.00	66.60	58.70	88.60
October.......	2.00	24,486	419.85	6.75	426.60	5.67	40.98	16.75	1.50	2.00	66.90	493.50	58.30	55.70	86.44
November.....	1.92	14,782	293.91	24.46	318.37	11.68	32.83	65.72	8.40	.....	118.63	437.00	50.30	33.80	72.81
December.....	1.96	10,698	226.72	45.17	271.89	4.34	40.96	107.38	22.49	10.27	185.44	457.33	47.20	23.40	59.45
January.......	1.96	4,244	105.74	160.90	266.64	.92	18.59	94.18	63.09	15.58	192.36	459.00	40.10	9.00	58.09
Total.........	1.87	112,928	1,987.59	246.47	2,234.06	70.43	197.96	292.12	110.41	28.85	699.77	2,933.83	56.82	38.49	76.15

First crane commenced operation July 13th.

The detailed division cost for the year is given for Pedro Miguel as follows:

Concrete (495,037 cu. yd.):		
Cement	$1.5365	
Stone	.8242	
Sand	.3729	
Mixing	.1771	
Total cost of concrete		$2.9107
Large rock (2,765 cu. yd.)		$1.1483
Masonry (497,802 cu. yd.):		
Concrete	$2.8945	
Large rock	.0064	
Forms	.4387	
Placing	.3118	
Reinforcements	.0367	
Pumps	.0343	
Power	.0454	
Maintenance of equipment	.1723	
Plant arbitrary	.6847	
Division expense	.0792	
Total division cost		$4.7040

The division costs per month since June 30, 1911, have been as follows:

Month, 1911.	Cost per Cubic Yard of Concrete.			
	Pedro Miguel.		Miraflores.	
	Plain.	Reinforced.	Plain.	Reinforced.
July	$5.82	$6.26	$4.93	......
August	5.63	8.74	4.45	$11.17
September	6.08	11.91	4.41	16.32
October	5.26	8.85	4.50	21.39
November	5.80	9.94	4.89	23.76
December	6.27	8.98	5.06	14.45

It is obvious from the tables that a large percentage of delays in placing concrete at both Gatun and the Pacific locks is chargeable to forms. At times the forms are filled so rapidly that it is difficult to keep them ahead of the placing, but the greatest loss of time is occasioned by the amount and complication of forms for the electrical tunnels, conduits and machinery rooms near the tops of the walls. The records given for the Miraflores plant are not representative, as the entire plant is not in operation.

USE OF CONCRETE IN THE FOURTH AVENUE SUBWAY.

By Frederick C. Noble.*

The Fourth Avenue Subway, Brooklyn, is an example of the use of reinforced concrete on a large scale. The portion now under construction extends about four miles from the Brooklyn end of the new Manhattan Bridge, over which it is intended to connect with a subway system in Manhattan. The route, Fig. 1, lies through Flatbush Avenue, Fulton Street, Ashland Place and Fourth Avenue, to Forty-third Street; in a direction generally south. From here it is proposed to extend it in future by two branches to the southern limits of the borough. The subway is being built for the City of New York, under the supervision of a state commission, with Mr. Alfred Craven as the chief engineer. The work was divided into six contract sections, which were let in November, 1909, at an aggregate price of about $15,-000,000. Construction is now nearly finished.

The structure normally has space for four tracks; two for local and two for express service. These are increased to seven and eight in places where connection spurs, for future extensions, are provided in two levels to avoid turnouts at grade. There are six local and two express stations, with platforms long enough to accommodate ten-car trains.

The excavation was principally in a moraineal deposit, consisting of sand with more or less gravel and boulders. No rock was found anywhere on the line. Some quicksand was met near the middle of the route, where the subway traverses ground filled in over the bed of an old salt marsh. Ground water was encountered at or near tide level and was controlled by pumping. Excavation was usually carried on under covered roadways, but in the extension of Flatbush Avenue open excavation was permitted.

In connection with the work it was necessary to underpin

* Division Engineer, Public Service Commission, Brooklyn, N. Y.

about forty buildings with masonry piers to sub-grade in advance of the excavation, where it approached close to and below their foundations. It was necessary also to temporarily support the elevated railway on Fulton Street and to drift a crossing under the present subway in operation in Flatbush Avenue. Since Fourth Avenue lies along the foot of a considerable drainage area, sloping from Prospect Park, it was necessary to depress or intercept many cross-sewers carrying a heavy storm-flow, and to provide under-grade crossings at intervals. About six miles

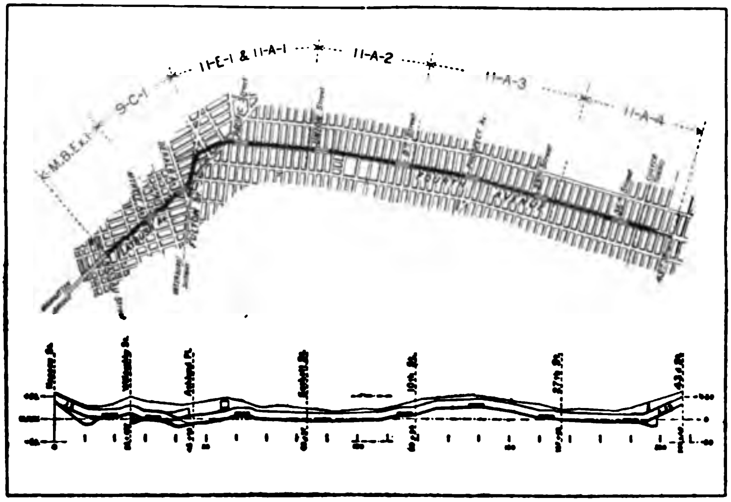

FIG. 1.—PLAN OF FOURTH AVENUE SUBWAY, BROOKLYN, N. Y.

of sewers, of all sizes up to 9½ ft. diameter, were thus built or rebuilt.

The typical four-track section is shown by Fig. 2. The roof, sides, intermediate walls and floor, where below ground water, are reinforced with bars ranging between 1¼ and ⅝ in. square. The design provides for a uniform live load of 300 lb. per sq. ft. at the street surface. Waterproofing is used at stations on the roof and sides and generally below ground water on the sides and floor. Elsewhere the concrete is relied on for sufficient protection against seepage.

The amount of concrete required is somewhat over 400,000

cu. yd. The usual proportions are 1 to 2½ to 4½. Much of the excavated sand is suitable for concrete and is used accordingly. The remainder is washed or dredged sand from the north shore of Long Island. The aggregate is mostly washed or dredged gravel from the same source, usually under 1 in. and well graded in size. At times when gravel is not readily obtainable, cobbles and fragments of boulders are crushed and used. The concrete is mixed rather wet to facilitate its flowing around the reinforcement, as this is generally spaced quite close.

Reinforcing rods were required to be deformed. Trans-

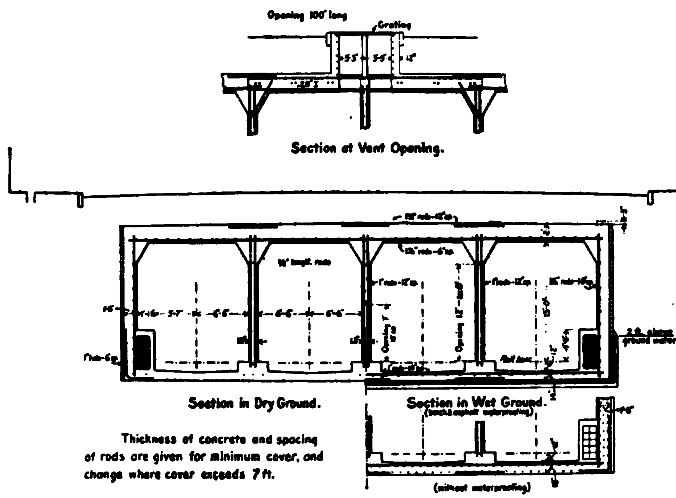

FIG. 2.—TYPICAL FOUR-TRACK SECTION OF SUBWAY.

verse and longitudinal rods were wired at their intersections (Fig. 8). For this purpose, special devices of bent wire were used; these served at the same time to space the rods, or to hold them at the proper minimum distance from the forms. Where unusually long spans or concentrated loads occurred, the roof was of girder and jack-arch construction. To hold the thin covering of concrete under the lower flanges, special clips or hangers of twisted wire were attached at intervals by bending them over the upper sides of the flanges. Sometimes a strip of coarse wire mesh was wrapped loosely around the flanges to serve the same purpose. In a few instances, where the concrete

under the flanges tapped hollow, it was chipped off around the edges of the flange, and a strip of wire mesh was attached and covered with mortar blown in place by means of a cement gun.

Concreting was permitted throughout the winter, except on the very coldest days. Materials were heated by fires or steam coils, and the freshly laid concrete was covered with salt-marsh hay and tarpaulins. Salamanders were placed under the roof forms in some places. On account of the varying conditions of mixing and degree of exposure, it was impracticable to devise a set of rules to fit all cases; but the criterion was held to be the temperature at which the mixture could be deposited in the forms.

Roof forms were struck as soon as the temperature conditions and setting qualities of the particular brand of cement used would permit. As this was a factor limiting progress, especially in the case of large steel forms, it was desirable to strike them as soon as practicable. In summer they were struck after 80 hours, and in exceptional cases even at 50 hours, but in winter it was sometimes necessary to wait a week or 10 days before striking.

The methods of mixing and handling concrete vary between the different sections and are described briefly as follows:

Section M. B. Ex. 1, extending from Nassau Street through Flatbush Avenue to Willoughby Street, is being built by Smith, Scott & Co., contractors.

Materials for concrete are brought on the work over a tramway extending along each side of the cut from a dock under the Manhattan Bridge. Sand and gravel are loaded at the dock in side-dump cars, drawn by 20-ton dinky locomotives. The cars are run up an incline and dumped into bins over the mixer, which is located about centrally on the section. This is a 1½-yd. machine, operated by a 30-h.p. electric motor. The mixture is discharged at the track level into bottom-dump cars of 1-yd. capacity, in trains of 3 or 4 cars, hauled to the point of deposit and dumped in frame chutes through which it slides to the forms.

As the sides of the cut were almost self-sustaining and as the contract terms allowed open excavation on this section, very little cross-timbering was necessary; a condition that greatly facilitated the erecting and moving of forms. The forms were

of the collapsing pattern, with stiffened steel sides and wood top to suit the varying widths of roof. The floor having been laid for the entire width of structure, and the reinforcement placed for the sides and intermediate walls, the forms were moved into place, one 20-ft. section at a time, traveling on tracks laid on the floor. In this way it was possible to lay concrete for the walls and roof over four tracks in sections 20 ft. long at a continuous operation. These forms were very similar to those used on one of the lower sections, in which connection they will be more fully described.

Where structural roof framing was substituted for rod con-

FIG. 3.—COLLAPSIBLE JACK ARCH FORMS.

struction, centering (Fig. 3) was used to turn the jack arches. These were of angle-stiffened plate, with an adjustable center strip of wood for varying the span, and were set in position on falsework. The span was maintained by turnbuckle ties, which also served to draw in the sides on striking the forms. After striking, each section was lowered with a winch.

Section 9–C–1, extending from Willoughby Street along Flatbush Avenue and Fulton Street to Ashland Place, is being built by William Bradley, contractor.

Materials for concrete are delivered at a large yard alongside the subway between Third and Sixth Streets, where the contractor has a cement storehouse and a stable for his horses.

Teams are employed for moving all materials. Concrete was mixed principally by a gravity mixer located in the cut. Storage bins for sand and gravel were placed just below the street level. These fed 4 loading hoppers, each of one-bag-batch capacity. After moistening, the contents were dropped successively through three mixing hoppers, receiving the full dose of water in the first one. The middle hopper was slightly offset from the other two. The mix was discharged at the bottom level into cars which were run onto a cage, hoisted above the street surface, and dumped into an overhead bin. From the bin it was almost immediately discharged through a gate into tight rear-dumping steel carts of about $1\frac{3}{4}$-yd. capacity, which were teamed to the point of deposit. The mixture was dumped on hopper platforms and conveyed through 8-in. telescopic chutes to the forms. The opening at the bottom of the hopper was closed by a spatula until the entire load was cleared.

Most of the structure on this section is of steel bent and jack-arch construction, so that no unusual form-work was required. The reversion to this older type of design was at the request of the contractor, and partly because of the concentrated loadings brought by the elevated structure and building foundations to be supported permanently on the roof.

Sections 11–E–1 and 11–A–1, which together form one contract section, extend from Fulton Street, through Ashland Place and Fourth Avenue, to Sackett Street, and are also under contract with William Bradley.

The methods of mixing and placing concrete are the same as those described for this contractor's adjoining section. The form of construction is also similar, except where the line passes by a curve under the present subway in Flatbush Avenue. Here the design is of massive concrete arches, one for each track, without reinforcement (Fig. 4).

Section 11–A–2, from Sackett Street along Fourth Avenue to Tenth Street, is being built by the E. E. Smith Contracting Company.

Materials for concrete are stored in a yard beside the Gowanus Canal at Third Street, and are distributed to the work by mule teams. Concrete is mixed in the roadway, directly over the point of deposit, in five small readily-shifted drum mixers of

½-yd. capacity, each operated by a 15 to 25-h.p. electric motor. The mixture was discharged and conveyed to the forms through 10-in. sectional chutes of 12-gauge iron, with a funnel at the top.

Generally, the floor and walls of the westerly two tracks were built first, then the floor and walls of the easterly two; after which the roof was carried across all four tracks in sections about 30 ft. long. Five such sections would be in progress at one time.

The necessity for maintaining the cross-bracing of the trench precluded the use of large forms such as were used on certain of the other sections. Forms were made of 2-in. lumber, dressed all sides, and with dapped edges. They were assembled in well-

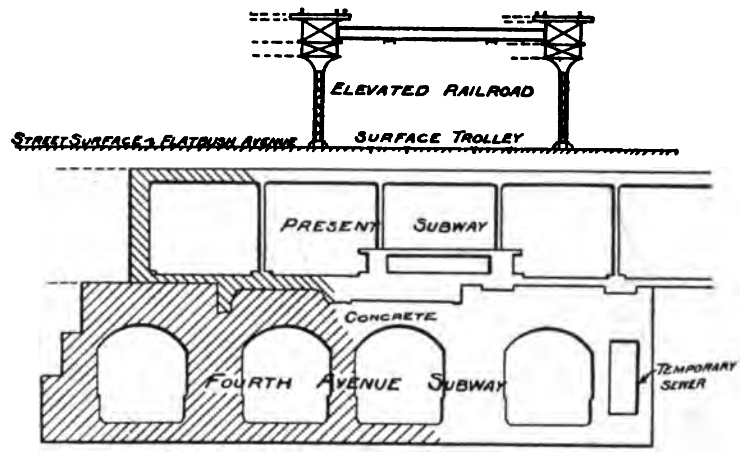

FIG. 4.—TYPICAL SECTION THROUGH SUBWAY AT TUNNEL UNDER FLATBUSH AVENUE.

braced panels and could be used many times. Jack-arch forms (Fig. 5) were made of 16-gauge iron, nailed on wood ribs and were suspended in position by long bolts.

On this section the usual waterproofing on sides and bottom below ground water level is omitted. In such situations the concrete is locally made richer, 1–2–4, and the longitudinal reinforcement is doubled (Fig. 2).

Section 11–A–3, from Tenth Street along Fourth Avenue to Twenty-seventh Street, is under contract with the Tidewater Building Company and Thomas B. Bryson.

Materials for concrete are brought on scows to a dock at

the foot of Nineteenth Street, from which a 3-ft. tramway leads to the cut and through it in both directions. The sand and gravel are transferred from the scows to overhead bins by a crane and clam-shell bucket. The bins discharge through gates into 4-yd. side-dump cars. These are hauled by a 20-ton locomotive to the main storage bins, of about 3,000 cu. yd. capacity, situated in the cut near Nineteenth Street near the mixing plant. Belt conveyors run under the gates of the bins and take the material to a bucket elevator which raises it to bins over the mixer. The

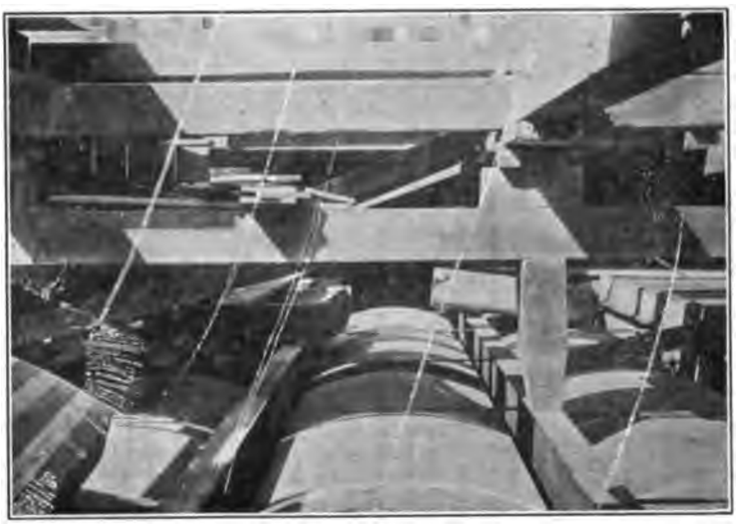

FIG. 5.—METAL JACK ARCH FORMS.

overhead bins feed into measuring hoppers, where the cement is added, and thence into the mixer, which is a 1-yd. machine mixing a 5-bag batch. Conveyors and mixer are electrically driven.

Taking advantage of the firm character of the ground and the great width of the avenue, a method of excavation was adopted that dispensed with cross-bracing; thus leaving the cut (Fig. 6) unobstructed and making it feasible to construct large sections at one time. The floor of the two middle tracks was laid first, and this was followed by the floor and duct bench of each of the two outside tracks. Concrete was brought from the mixer for this part of the work in 2-yd. side-dump cars. On the comple-

tion of a section of the bottom, steel forms were moved into place, and the sides, intermediate walls and roof were concreted continuously in a long section.

The steel forms (Fig. 7) were of the collapsible type, made of $\frac{3}{16}$-in. plate and stiffened with shapes. They were composed of 5-ft. units, bolted together in sections 40 ft. long, and traveled on rails. Four such sections, one for each track, made up a set of forms; of which two were in use generally on different parts

FIG. 6.—METHOD OF OPEN CUT EXCAVATION.

of the work. The forms were struck, each section separately, by means of a hand tackle arrangement that swung the top leaves down around hinges at each side, and drew in the tops of the side-panels automatically. The vertical reinforcement (Fig. 8) for the next section being set up, the forms were then pulled ahead, one 40-ft. section at a time, by a locomotive, into its new position; an operation that could be performed in a very few minutes. As each section was advanced it was adjusted to exact position and connected to its neighbor by long bolts and

pipe separators. The adjustment was effected by the jack-screws and sliding axles of the trucks; affording a range of movement both vertically and horizontally. The reinforcing bars of the roof were then laid in position.

For concreting the roof, the mixer discharged into trains of 4 or more 2-yd. bottom-dump buckets on flat cars, which were pushed by a locomotive over the tramway through the cut to the forms. Here the buckets were lifted by a locomotive crane

FIG. 7.—COLLAPSIBLE STEEL FORMS FOR ROOF AND SIDES.

(Fig. 6), and dumped on a hopper raised about 12 ft. above the forms. From the hopper the mixture flowed through shallow troughs to the points required. When both sets of forms were joined together, an 80-ft. section, requiring over 600 cu. yd., could be concreted in about 15 hours.

Section 11–A–4, from Twenty-seventh Street along Fourth Avenue to Forty-third Street, is being built by the E. E. Smith Contracting Company, who are also the contractors for section 11–A–2.

Materials are delivered into bins in the contractor's yard at Thirty-first Street and Second Avenue, where there is also a cement storehouse of 30-cars capacity to serve both sections.

The methods of mixing and distributing concrete and the use of forms, are the same as those described for section 11-A-2.

All cement is inspected at the mills and shipped in sealed

FIG. 8.—VERTICAL REINFORCEMENT.

bags. The testing laboratory is in Allentown, Pa., in the Lehigh Valley district. Over half a million barrels are required. Under the specifications, preference is given to brands whose records show continued increase in strength over long periods. As a criterion of this quality, mortar briquettes are required to show a gain of at least 50 lb. between the 7 and 28-day tests.

THE USE OF REINFORCED CONCRETE IN HYPOCHLORITE WATER PURIFICATION WORKS.

By Walter M. Cross.*

During the year 1911 an experimental installation of the hypochlorite process for the approximate sterilization of the entire municipal water supply of Kansas City was so remarkably successful in diminishing the sickness and death rate in the city on account of typhoid fever as well as other forms of intestinal disease, that the Kansas City Fire and Water Board undertook the construction of a permanent building and apparatus for the application of this purification process to the water supply.

A separate building was constructed to make possible the satisfactory storage, handling and making of the solution of hypochlorite ready for mixing with the sedimented water. The building itself was designed by W. C. Root, an architect, and the apparatus for use in connection with the sterilization process was installed under the direction and supervision of Burton Lowther, engineer in charge, and S. Y. High, superintendent of the Water Works Department.

The apparatus for the handling of the hypochlorite and the supports for it are of reinforced concrete. It is to be observed that no other material is so well suited for use in connection with this sterilizing agent as good concrete for the reason that all other materials that are capable of oxidation are promptly attacked by the hypochlorite solution and become rapidly deteriorated. The prime consideration with regard to this class of installation is to employ such methods of construction and to use material that is so permanent in character as to obviate the necessity of repairs which would force the discontinuance of the application of the sterilizing agent even for an hour.

The basement of the building is used for storage of the reagent that is kept in reserve. The main floor is used to house the dilution tanks and the feeding devices, while on the floor above is placed the tank in which the hypochlorite is reduced to paste

*City Chemist, Kansas City, Mo.

of a creamy consistency before being delivered to the dilution tanks beneath. This pasting tank, 3 ft. in diameter and 4 ft. high, is provided with a stirring device carrying two rather heavy rollers, disposed horizontally at its lower end. The rollers clear the bottom of the concrete tank only by a fraction of an inch, thus insuring the mashing and disintegration of all of the small lumps that are invariably present in commercial calcium hypochlorite. Owing to the fact that the action of the reagent on bronze is to form on the surface of it a fairly insoluble and protective coating of metallic carbonate and oxychloride, that metal appears to be the most available for use on all bearings and stirring or disintegrating devices that come in contact with the solution. Leading from the concrete pasting tank are pipes so arranged that the contents of the tank may be discharged into either of the large dilution tanks on the floor beneath. The outlet of the pasting tank is placed at a considerable distance above its bottom so as to avoid the possibility of drawing off with the paste any fragments of considerable size. The pipes carrying the paste are so arranged as to be readily cleaned in a few minutes in the event that they become clogged. Ultimately they are sure to become clogged if they are not occasionally cleared because of the formation in them of carbonate from carbon dioxide absorbed from the air.

The dilution tanks are hexagonal in form, 9 ft. in maximum diameter and 7 ft. high; the walls are 6 in. thick. Although the difficulty experienced in properly disposing the reinforcing metal in the construction of a hexagonal tank is much greater than is the case in the building of a round one, the hexagonal tank is to be preferred on account of the fact that in a round tank a rotary stirrer does not produce nearly such thorough agitation and mixing of the solution of hypochlorite as the same stirrer can do in the hexagonal tank. The paste is mixed with water in the dilution tanks until a uniform solution of a strength of 2 per cent occurs. The use of the two tanks makes it possible to accurately adjust the strength of the solution in one dilution tank while the contents of the other are being utilized. The dilution tanks are placed on supports high enough to permit the use of a gravity feed to the orifice box which is placed on the floor of the room housing the big tanks.

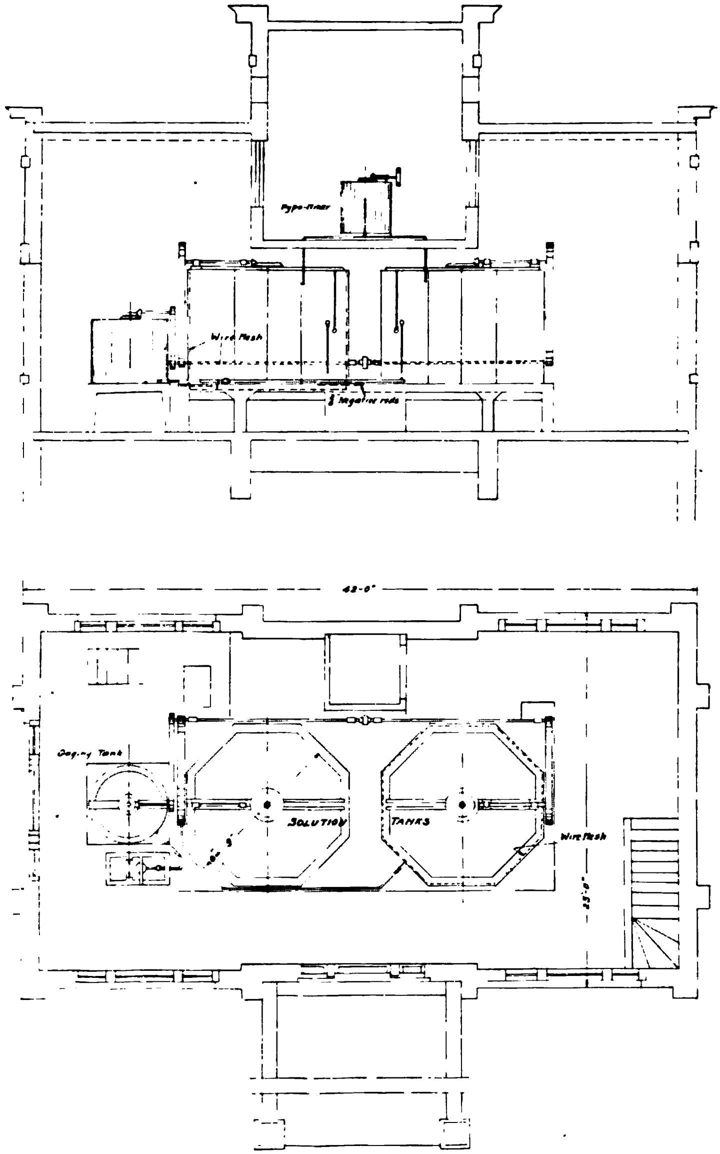

FIG. 1.

Bronze pipes, 1½ in. in size, so arranged as to be readily cleaned in the event of stoppage, connect the dilution tanks with a gauging tank, 4 ft. in diameter. This gauging tank contains a float, scale and pointer so arranged that the man in charge can accurately check the speed of outflow of solution from the orifice box into the big water main carrying the entire city water supply from the settling basins to the pumps. The solution passes through the gauging tank to the orifice box. Each division on the gauge represents 1 gallon of the hypochlorite solution.

The orifice box is oblong in shape and carries a float of about 250 cu. in. displacement. The float operates a valve which, by either opening slightly or closing, maintains the hypochlorite solution in the orifice box to a constant level. One end of the orifice box is of plate glass to enable the operator to see at a glance that the solution is filling the box to the proper height. Attached to the plate glass and covering a hole in it, is a hard rubber disc having near its periphery several slits, the adjustment of which represents the size of a stream of the 2 per cent hypochlorite solution that will be the proper amount to treat the quantity of water passing through the main. All movements of the hypochlorite solution after its preparation are by gravity. Ample opportunity for the hypochlorite after its addition to the water to react with any putrescible organic matter and germs, is afforded during the time in which the water passes through the centrifugal pumps, the flow line and a small storage basin at Turkey Creek before it is pumped to the domestic water users.

All of the stirring devices are run by an electric motor belted to a line shaft carrying clutches so placed as to make possible the running of any one of the stirrers whether or not any of the others are running.

The principle involved in the construction of practically all hypochlorite installations for the purification of water by the oxidation of germs and putrescible organic matter in municipal water supplies is substantially the same as that in Kansas City. Concrete, usually reinforced, is universally used in the construction of all permanent apparatus for the preparation and solution of the hypochlorite for mixing with the water to be purified.

A fairly good idea of the disposition of the various parts of the purification installation is given in Fig. 1.

DESIGN AND CONSTRUCTION OF THE ESTACADA DAM.

By Hermann V. Schreiber.*

Located in the northwestern section of the country, where fuel costs are high, it is natural that the Portland electric companies should have early appreciated the abundant stream flow characteristic of the region, which results from the high precipitation on the western slopes of the Cascade Mountains and in the Willamette Valley and which was rendered available for their power purposes by the successful development of high tension electric transmission for the distances involved in delivering such hydro-electric power to their markets.

The first plant developed to serve Portland, and one of the first water-power transmission plants on the coast, is located at the falls of the Willamette, adjacent to Oregon City on the Willamette River, which stream flows through Portland and empties into the Columbia River about six miles below the center of that city. Some years later, as the community rapidly grew in size and industrial activity, a combination plan was projected by interested parties for the construction of a railway and power development on the Clackamas River, based upon an available power site some forty miles east of Portland, and this development has since been completed and, with the other railway and lighting interests, has been included in a consolidated property known as the Portland Railway, Light and Power Company, which at present supplies practically the entire electrical market in Portland, including the local and interurban railway systems.

Following the consolidation, the rapid increase in the power requirements of the community indicated the need for immediate additions to the generating capacity, and Sellers and Rippey, Consulting Engineers of Philadelphia, were retained by the financial interests in control of the consolidated property to investigate and report upon certain features of the existing plants and

* Sellers and Rippey, Consulting Engineers, Philadelphia.

the power possibilities and costs of certain new developments which were suggested for consideration from time to time during the investigations. Because of the exceptional water-power possibilities of this region, the number of alternative locations presented for consideration was greater than would ordinarily come within the radius of economical transmission to a large city, but attention was chiefly directed to the Clackamas River, upon which the company already owned the Cazadero development, the electric railway system and the transmission lines connecting the generating station with Portland.

This stream has its source in the forest-covered western slopes of the Cascades and the snow-covered peaks adjacent to Mount Hood. The run-off resulting from the high precipitation peculiar to this section is distributed in a remarkably uniform stream flow which, because of the favorable geological formation, is maintained even during the long dry summer season. Consideration was primarily given an available site for a large new development upon property already owned by the company on this river above the existing Cazadero plant, but investigations extending over the thirty miles of the river downstream to its mouth revealed a site capable of economic development, which, though it did not offer as large capacity as the upper site suggested, was found to be worthy of recommendation for the company's immediate consideration, because of the considerably lower cost involved and sufficient size to meet the immediate power requirements. This site, by far the most attractive on the lower river, is a short distance below the company's town of Estacada, about 3½ miles below the original development at Cazadero, and offered the advantages of a reasonable head for direct development without long flumes, accessible railway connections and several incidental advantages from a construction standpoint. After some delay, the necessary property was acquired and executive decision given that the construction proceed without delay. Within a distance of a few hundred feet several possible dam sites were available, giving the same head, with advantages and disadvantages peculiar to each, making the proper selection one of considerable difficulty. Fig. 1. gives a general view of the Estacada dam, a plan of which is shown in Fig. 2.

FIG. 1.—GENERAL VIEW OF ESTACADA DAM.

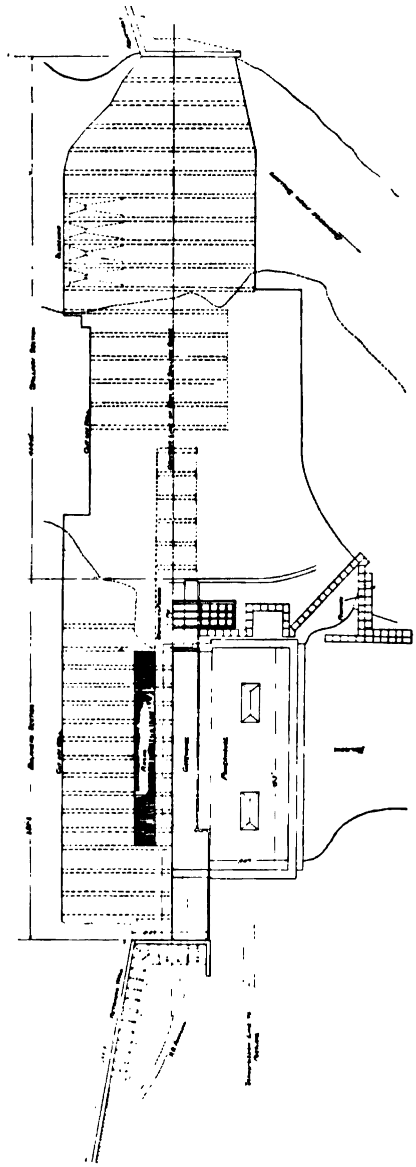

FIG. 2.—PLAN OF DAM AND POWER HOUSE.

It is of considerable interest at this point to consider further the character of the country, its geologic history and bed-rock formation as affecting the selection of site, preparation of foundations and design and construction of dam.

The mountains which form the source of the water supply include several extinct volcanic peaks, the eruptive discharge from which has covered the country for miles around with various forms of volcanic debris of such porous and uncertain nature that it introduces serious obstacles to satisfactory hydro-electric developments of any considerable magnitude, even when they are limited to the use of a low dam and extended canal such as that which has been installed at Cazadero. In order to properly study the situation and secure all possible information relative to the formation of this material which now constitutes the "bed rock" a careful examination of this section was made on request by Mr. J. S. Diller of the U. S. Geological Survey, who in his reports on the subject made the following statements:

The volcanic breccia (bed rock) is made up of unassorted angular fragments of lava andesite and basalt of various colors ranging in size from dust particles and grains of sand to large rock fragments many feet in diameter. This fragmental material was blown by explosive eruption from the volcanic craters higher up on the range and fell upon the mountain slopes where it became so saturated with water from the copious rains accompanying the eruptions that it flowed in great steaming sheets from the Cascade Range to the gentle slope of the plains, in much the same way as similar material flowed down the old stream channels on the western slope of the Sierra Nevada in California and covered the early and often rich deposits of auriferous gravels.

Sheets of solid nonfragmental lava forming part of the bed rock and outcropping on the slopes of the canyon occur within and between the great sheets of volcanic breccia. Some of the lava sheets are basalt, others are andesite and they are usually less than 30 ft. in thickness. The basalts are generally very porous and gray or dark. The andesites are often reddish and porphyritic with white crystals of feldspar.

The depth to which these sheets of volcanic breccia and lava extend cannot be readily determined but it is certainly hundreds of feet and may be, as it is along the Santiam and McKenzie River canyons, over a 1000 ft. in thickness.

Nearly vertical dikes of basalt cut up through the sheets of volcanic breccia and lava and outcrop on the surface. These dikes in some places have a well developed columnar jointing which divides the rock into columns. In the case of the dikes the columns lie horizontally and extend across the dike. In the lava flows the columns are vertical, but in all cases the columnar

joint cracks are limited to the dike or lava sheet and do not extend into the adjacent rock nor make an opening of great extent. There is, however, another set of parallel joints, the open cracks of which cut up through the volcanic breccia and sheets of lava about vertical in a direction approximately parallel to the course of the canyon. Such joints may be of considerable extent and form important openings for the circulation of water; Such joints may be expected and should be carefully looked for where the rock is covered with soil or gravel. It is especially significant that the dikes are approximately parallel to these joint cracks and suggest that the joint cracks may extend to great depths.

The conditions that confront the engineer along the Clackamas River in the volcanic breccia plain region are very much the same as will be found all along the western foot of the Cascade Range from the Columbia River in Oregon to Feather River in California, one of the most important water-power belts in the United States, and the successful solution of the problem which it presents at one point will greatly facilitate the work elsewhere.

The original development at Cazadero included a rock fill timber crib dam covered on the up-stream slope with a large quantity of surface soil and gravel, used to reduce to a minimum the leakage beneath the structure. The dam creates less than one half the operating head, the balance resulting from the extension of a head race canal and flume about 1⅔ miles long to the generating station, the tail water of which is much lower than the base of the dam because of the intervening slope of the river. This represents a common method of development in the West, where first cost, rather than low operating, maintenance and depreciation charges, has in the past largely governed such projects. This development was well advanced when taken over by the present owners and as completed by them utilized conditions advantageously, although it possesses in a degree the inherent disadvantages of such construction in that its peak load capacity is limited by the forebay pond capacity and the leakage under the dam and from the canal tends to reduce the dry season plant capacity. The present owners have however succeeded in reducing the leakage under the dam to a trifling amount.

Considering the magnitude and permanence of the consolidated property, it was felt desirable in planning extensions to endeavor to provide for direct power developments, creating the entire head by a concrete dam affording large storage reservoir capacity and locating the plant directly at the dam without intervening flume or canal, thus permitting almost indefinite peak

load capacity to be developed. This method of development involves higher dams and consequently demands greater assurance concerning the security of foundations and before asking Mr. Diller to report upon the geological formation core drill investigations were started to determine the characteristics of the underlying material. His report, as will be noted, strongly confirmed our own conclusions as to the importance of this problem and we continued our extensive explorations with core drills, going as deep as 250 ft. at times, and not content with a partial examination, we made investigations at considerable cost and trouble in the river bed as well as upon both banks.

SITE.

At the first new site considered for the development of power on the Clackamas River above Cazadero it was proposed to create 135 ft. head at a direct connected plant, providing large pond storage which would be of considerable value to all the plants existing or constructed at any future time on the river below this point.

At the Estacada site below Cazadero it was proposed to develop power under 83 ft. head, utilizing all the available fall in the river in the 3½ miles below the original Cazadero power station and providing pond storage which would be of great value in carrying the daily peak load.

The investigations for the higher head development at the upper site were started before the lower property was acquired and were under the direct supervision of the field engineer, Mr. Shirley C. Hulse, under our direction. The very thorough study of the situation there gave considerable data for use in developing a method of treatment to insure as far as it might be practicable an absolute cut-off across the canyon which would prevent any undue leakage or erosion of the river bottom after completion of construction, and the results of this work were immediately applied to the Estacada construction, saving much time in the preliminary engineering there.

When the company was ready to proceed with this construction the work was very urgent. The general preliminary investigations covered both sides of the river for a distance of several hundred feet; well drillings were made without finding any

radical difference in the character of the bed rock, except that the first and the last of the several sites considered were found to be underlaid with a soft clay formation which might have necessitated a reinforced mat or piling to support part of the structures. Of the four sites here considered, the third one was selected as offering an island which it was expected would considerably reduce the amount of material required and at the same time make the construction much more convenient by facilitating the diversion of the stream through the rainy season.

Further investigations were then made of the actual conditions on the island with the result that a large ravine was found to extend parallel to the thread of the stream, which being filled with debris necessitated its entire excavation. As the construction proceeded the left end of the island adjoining this ravine was found to contain a layer of clay sloped in such a way as to endanger its stability if loaded with any of the dam superstructure. This was also removed and with the removal of another section of the island on account of the great uncertainty of the effectiveness of any device against leakage there was little left of the island and no expense was saved by its use in the construction.

CUT-OFF WALL.

The study of the foundation conditions on the Clackamas River was evidenced by the investigations and test pits, together with rough pressure tests, Mr. Diller's expressions relative to the geological formation, and the general hesitancy among engineers with respect to constructing high masonry dams upon such foundations confirmed the original belief that somewhat novel methods must be adopted to give reasonable assurance concerning the integrity of the work to be constructed. Moreover, it was evident that the nature of these methods should be such as to permit demonstration of their efficacy before any large investment in the construction should be made, and this involved some experimental work and expense which would not be involved on more substantial or satisfactory foundations.

It will be evident that impermeable foundations are desirable for the following reasons:

(*a*) To minimize the possibility of *upward pressure* under the

base of the dam superstructure. (This applies chiefly to solid dams.)

(b) To prevent percolation under the dam which might lead to sufficient erosion to involve *undermining* of the structure.

(c) To overcome any *structural weakness* due to the original geological formation and properly provide a support for the superimposed load.

(d) To avoid *waste of water* (the chief power asset of the company) from the reservoir, through, under or around the dam, instead of through the turbines, thus providing K.W.H. for sale.

To satisfy these conditions with existing foundations of the character described in Mr. Diller's reports required a departure from any methods heretofore used of which we had knowledge and in studying the problem it appeared that the only safe program must practically provide for actually changing the structural character of the underlying formation. It occurred to our chief engineer, Mr. S. Howard Rippey, who had spent some time on the ground, that the most promising method would be the solidification of the porous material by the introduction of cement grout under pressure, but that the success of the work must be susceptible of demonstration by actual test before the superstructures should be started. Thorough inquiry failed to disclose any precedent for the use of grout for the general treatment of foundations, although it was found that cavities in limestone rock under the New Croton Dam had been filled with grout, much as a dentist would fill a cavity in a tooth. The grouting method had also been used in filling back of lining walls in tunnels, etc. Notwithstanding the absence of precedent, it was decided to proceed with a grouting scheme and a program was outlined for preventing leakage of water from the reservoir created by the dam, under or around the dam, to the low tail water level below the dam and the experiments which should be made to properly demonstrate the efficacy of the method before the complete development should be undertaken were prescribed. The general idea provided for drilling a double line of holes of an average depth of say 50 ft. under the heel of the dam across the entire valley to and under the shore abutments and the subsequent forcing into each of these holes of grout of such consistency as to percolate through the entire substructure and so permeate it as

to solidify it absolutely throughout the entire length of the superstructure, thus making the foundations absolutely solid and the equivalent of a deep cut-off wall.

It was recognized that experiments would be necessary to determine the proper spacing of the grout holes and their depth, which would ensure sufficient diffusion of the grout through the varying material encountered to create a continuous impermeable barrier, thus preventing seepage of water from the reservoir under the hydrostatic head which would be created by the dam.

After drilling the double line of holes, the program contemplated the test of each hole with water pressure, a record being kept of the quantity escaping and the pressure applied to each hole. This water pressure test also provided for washing out the interstices ready for the reception of the grout. Upon the completion of the tests, the cement grout was to be pumped into the holes under pressure and after allowing time for hardening, a third line of holes was to be drilled midway between the first two or outer lines and tested with water pressure. The idea was that if water pressure be applied to the center holes at or slightly above the hydrostatic pressure to which this rock would be subjected by the water in the reservoir after completion of the dam and no appreciable leakage occurred, we should feel reasonably certain that the cement grout had proven effective in making the entire foundation impermeable.

Before purchasing apparatus to handle the thin cement grout communication with several pump manufacturers showed that in so far as the manufacturers' guarantees were concerned, choice was limited to two standard makes of hand-operated diaphragm pumps which would require 8 men on the handles to develop a pressure of 100 lb. per sq. in. Only one power diaphragm pump was offered by the manufacturers who would assume no responsibility for its successful operation. It was found that in both this and foreign countries plunger, diaphragm and centrifugal pumps had been successfully used, but after careful study of all the commercial pumps available it was decided that the use of compressed air would be more effective, flexible and economical than any mechanical pump. Having eliminated the question of pumps it was found that several different types of compressed air tanks were available; some being provided with different numbers and shapes of blades which were revolved in

the tank for the mechanical mixing of the cement grout while in others the grout was mixed entirely by the circulation of air.

Interviews were had with a number of contractors and engineers who had had occasion to use the different types of machinery and it was finally decided to purchase the Canniff Pneumatic Grout Mixer and Injector. This machine, Fig. 3, consists of a plate steel cylinder with a conical shaped bottom and flat plate

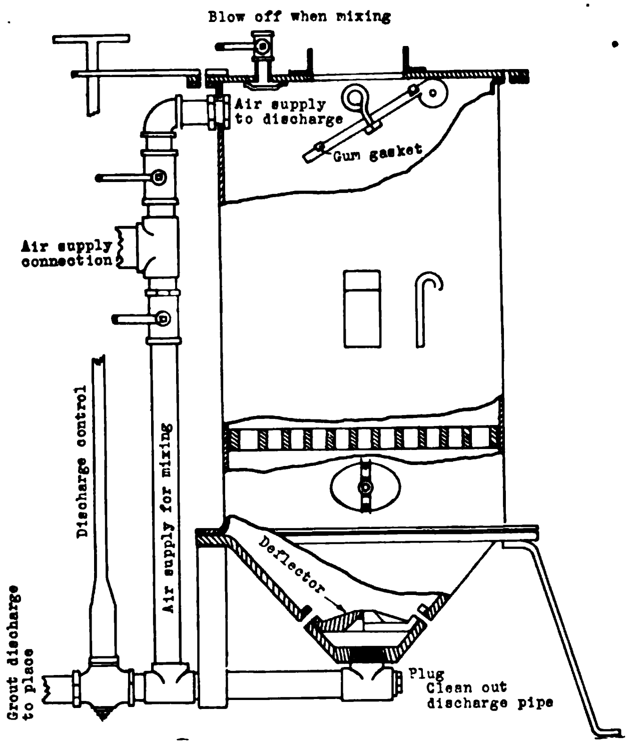

FIG. 3.—CANNIFF PNEUMATIC GROUT MIXER AND INJECTOR.

steel top which is provided with a smaller hinged circular lid opening inward through which the water, cement and sand are introduced. The tank is provided with a blow-off valve on top of the lid, also an air inlet pipe tapped into the side near the top, a grout discharge pipe tapped into the bottom and a by-pass pipe between the air and grout pipes. The valves are so arranged that when the cement, sand and water are placed in the tank the com-

pressed air is forced into the bottom of the tank through the grout pipe and having free escape through the blow-off valve at the top, the grout is thoroughly mixed by the passage of air bubbles through it.

When it is desired to discharge the grout the valves are so manipulated that the air is introduced near the top of the tank and the grout forced out through the bottom pipe, the pressure being limited only by the capacity of the compressor and the strength of the tank. It is possible to connect any number of these tanks on to one main grout discharge pipe, thereby insuring a continuous flow of grout and by a proper system of pipe connections any one tank may be cut out for repairs without interfering with the operation of the remaining tanks.

After experimenting on the Estacada foundations with both the shot and diamond core drills, an equipment of 7 Calyx shot drills was finally installed and operated at this development night and day for a period of about one year. These drills gave a hole varying from $2\frac{3}{8}$ to 8 in. in diameter depending upon the character of the bed-rock material.

Where possible, in advance of the core drilling 3-in. wrought iron pipe casings were set to a depth of from 4 to 6 ft. in the rock and grouted, the upper ends projecting about 1 ft. above the surface and being threaded to permit the attachment of the testing and grouting apparatus. While the original plan called for grouting in advance of any construction, the failure to start this portion of the work at once tended to retard the actual construction work and it therefore became necessary to construct a concrete cut-off wall 7 to 10 ft. deep and set the casings in this wall and drill the holes through the casings into the rock below where this was necessary. The cut-off wall was placed upstream from the foot of the dam and the construction of the dam superstructure was continued without interference.

The final report of Mr. Frank R. Fisher, resident engineer, for the Light and Power Department, Portland Railway, Light and Power Company, under whose supervision the Estacada construction work was completed, gives the following additional detail information relative to the conditions which existed and methods used in the grouting:

The rock mass is traced throughout with seams, very irregular in shape and size, extending in all directions with but slight continuity. They vary

from an almost imperceptible cleavage joint up to those having an approximate width of from one to two inches, the large majority observed measuring but a fraction of an inch. The seams are more or less choked with sand, gravel, small particles of rock, and other debris. No large crevices or faults were found at the site, nor observed in the vicinity of the dam.

On the whole, the rock gives indication of possessing fair bearing value, and while not what would be classed as hard, would probably offer considerable resistance to the erosive action of water, except under high velocities.

The plan was adopted of distributing drills over a wide area and each hole drilled was tested and grouted immediately on completion before putting down any other nearby holes. In grouting, the method usually followed was to make connection between the grout tanks and the casing by means of a flexible copper hose, and introduce the grout at the top of the hole; but in order to prevent, if possible, the rapid choking of the hole with cement, which frequently occurred, the method of introducing a pipe into the hole, and discharging the grout at various depths was tried out. For this purpose a 2-in. pipe, made up in sections, was used, and the operation was started with the same inserted to within a few feet of the bottom of the hole. When the hole gave evidence of tightening with the pipe in this position, one section was detached, usually 10 ft. in length, thus raising the outlet, and the operation repeated. At intervals a charge of water was shot in, to keep the pipe from plugging, and also to loosen up the cement that settled in the hole. This method of grouting through the pipes was given a thorough trial, but so far as could be observed it had very little advantage as to the amount of grout the hole would take, over the less laborious operation of introducing it directly at the top. The changing of the position of the pipes also interrupted the continuous flow of the grout, which it was desirable to maintain in order to accomplish the best results.

The consistency of the grout was varied to meet the different conditions, 1 part of cement to 5 of water appearing to give the best results, but the proportions tried out varied from 1 cement and 2 water to 1 cement and 15 water.

The grout was forced in under air pressure ranging from 50 to 200 lb. per sq. in., depending on the tightness of the hole, but the material at the lower depths would not tighten up beyond a certain degree. While it was desirable to use the higher pressures, in order to accomplish the greatest diffusion, it was not always practicable to do so, on account of it blowing out at the surface.

As the general nature of the rock had been investigated through the cores from the preliminary exploration holes, no special effort was made to preserve the same when drilling the primary holes. That from the proving holes, however, was carefully examined for any traces of grout, and while some very fair specimens of cement core were obtained, the amount was not large. This was probably due to the fact that most of the seams were small, and the cement entering therein would be ground to powder under the action of the bit. It is also probable that in many instances, at the time the proving holes were put down, the cement had not set sufficiently hard to core,

as the conditions arising from the various stages of the construction work, as well as the rate of progress to be maintained, made it necessary to follow on with the proving, within a short time after the grouting of the primary holes.

The pressure testing of the proving holes was important and the final plan adopted gave a direct pressure from tanks located above the proposed pond water level as best approaching the final conditions. After the piping was filled, a test run of ten minutes was taken, the rate of seepage per minute being averaged and recorded. Besides testing individual holes, several combinations were tested in order to determine if possible the extent of inter-communication beneath the surface. The combined seepage in this was found to be considerably less than the sum of the separate tests and in some instances was only about half. An additional refinement was introduced by the testing of each proving hole in some sections every 10 ft. in depth as it was being treated, thereby giving an indication of the effectiveness of the grouted cut-off wall at different depths. In some instances communication was found to exist between holes located as far as 70 ft. apart.

After the completion of the dam, and subsequent filling of the pond, two test holes were put down inside of the dam, approximately 30 ft. from the cut-off. One of these was located between buttresses Nos. 10 and 11 and the other between Nos. 12 and 13, opposite the weakest points in the grouted cut-off. After drilling to a certain depth, backflow was obtained from each, and the maximum height to which the water rose in an extended pipe, due to upward pressure, was within 13 ft. of the elevation of that of the pond. The results of these tests indicate that some upward pressure exists in the foundation material in this locality, which the mass of overlying rock is resisting.

The excavation of a supplementary cut-off trench at one point where grouting had previously been done offered excellent opportunity for observing its effects, as many seams were exposed and all of them proved to be well caulked with cement.

The data of quantities and cost of this portion of the work is also of interest.

DRILLING 555 HOLES, 34,038 FT., COSTING $1.55 PER FT. FOR DRILLING AND GROUTING.

Labor—	
Drilling	$0.58
Grouting	.18
Repairs	.17
Plant (drills and grout tanks)	.30
Cement	.12
Power	.05
Overhead (incorporated general plant)	.32
Total	$1.72
Less salvage	.17
	$1.55

In one section of the work the original pressure tests on the ungrouted holes frequently developed a seepage of over 100 gallons per minute per hole and communication existed between holes located over a wide area and surface leakage also appeared in many places. Where the original primary holes gave an average leakage before grouting of 80 gallons per minute, thirteen proving holes when drilled showed an average leakage of 7 gallons per minute. Another section, for the first 30 ft. of depth below the concrete cut-off the treatment appeared to be fairly successful, but at the full depth a test on a group of thirteen proving holes showed an average seepage of 6.4 gallons per minute. In still other sections the proving holes showed seepage varying from 2.2 gallons to 3.6 gallons per minute.

A total of 1942 bbls. of cement were used in grouting. The average depth drilled per hour was 1.32 ft., including time for moving, etc. The cement pumped into the holes varied from 3.32 bbls. to 50 bbls.

This treatment indicated the possibility of constructing what may be termed an effective cut-off without being literally impervious. There is however no assurance that this degree of success would result from similar work on another site with this kind of foundation.

DESIGN OF DAM.

Previous experience in the construction of hollow dams together with numerous studies of comparative costs and advantages inherent in hollow and solid dams prompted us to recommend the hollow dam for use in this instance particularly because of the character of the foundations. While several other forms of hollow dams are available and have been carefully studied and some have been built, the distance at which the work had to be directed together with the urgency of the construction strongly favored the adoption of the reinforced deck and buttress form of hollow dam, on account of the existence of an experienced construction organization which could be started at once on the work and push it to completion without delay.

The materials and stresses specified for use in this design were as follows:

Concrete—

Deck and apron	1:2:4
Buttresses	1:3:6

Reinforcement—Corrugated square bars 50,000 lb. elastic limit and 80,000 to 100,000 lb. ultimate strength.

Stresses—

Modulus of elasticity of concrete	1,500,000 lb.
Modulus of elasticity of steel	30,000,000 "
Compressive stress in concrete	500 lb. per sq. in.
Shear in concrete	75 " " "
Tension in concrete	0 " " "
Tension in reinforcement	15,000 " " "
Base pressures	100 " " "

The hollow dam of the design selected consists of a series of parallel walls or buttresses running parallel to the thread of the

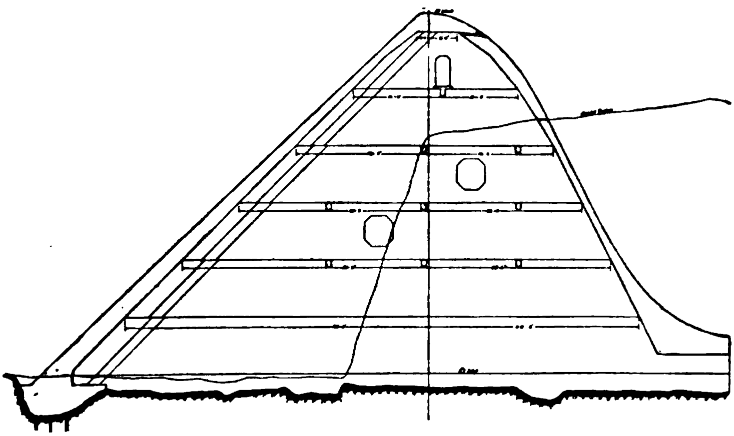

FIG. 4.—SECTION OF DAM THROUGH SPILLWAY.

stream, with an upstream covering or deck on one side and, on the spillway section, a downstream covering or apron which terminates at its base in a heavy curved bucket section designed to divert and discharge the falling water downstream parallel to the river bed. A section of the dam through the spillway is given in Fig. 4, and through the intake in Fig. 5.

The deck, which is inclined 45 deg. with the vertical, has a thickness that varies from 20 to 48 in. depending upon the depth. The apron which makes an angle of 30 deg. with the vertical also varies in thickness, being thickest at the crest and bucket but reduced to a thickness of 18 in. on the straight slope section. The reinforcement in deck and apron is laid horizontally 2 in.

from the under side of the slab with vertical bars on 24 in. spacing as an extra precaution against weakness in joints and to avoid temperature cracks. Hydrated lime, 30 lb. per cu. yd., was added to the deck, crest and apron to make the material less pervious. The extra cost for this material averaged 23 cents per yd. The buttresses spaced on 18 ft. centers are built in horizontal lifts of 12 ft. and vary in thickness from 15 in. at the top to 38 in. at the bottom, being also tapered along their length for equalization of pressure. Haunches with 18-in. seats for the deck slabs are pro-

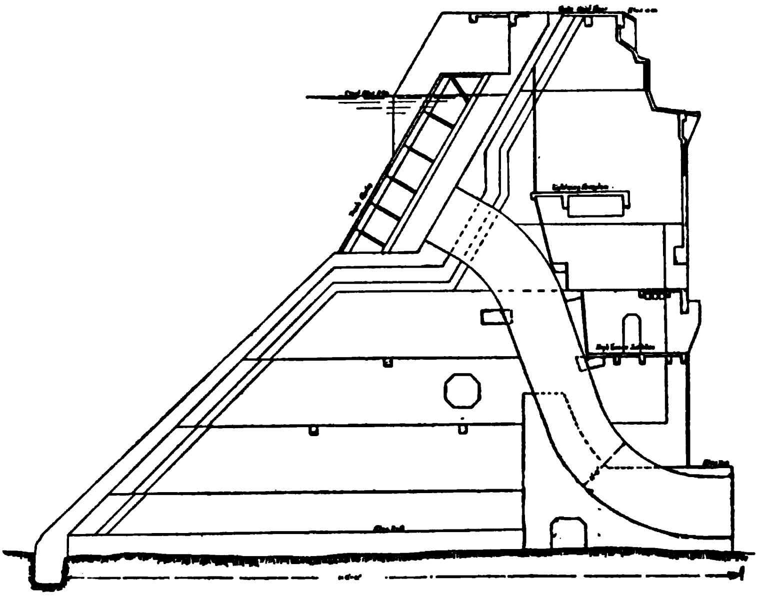

FIG. 5.—SECTION OF DAM THROUGH INTAKE.

vided at the upstream end and tongues extended between the slabs support the deck forms during construction and provide an opportunity to get a tight joint between the slab and the buttress. Because of the uncertain character of the foundations the footings were spread to reduce the base pressure to 100 lb. per sq. in. Additional reinforcement was also provided in the bottom lift to permit bridging any spaces of uncertain bearing value. The spillway section of the dam has a maximum height of about 86 ft. and the bulkhead section a maximum height of about 101 ft.

As constructed at Estacada the usual features were embodied in the spillway construction with the addition of sluice gates and the usual closing device in the left or main river channel. The spillway end abutment was of solid retaining wall section, a large part of it being built in gravel and clay, serving principally as a cut-off or core wall to prevent wash around the end of the dam. The right-hand or bulkhead end abutment was considerably longer, of reinforced retaining wall design.

The island which, as stated, was included in the development

FIG. 6.—METHOD OF CONSTRUCTION AND FORM WORK.

was found to be of such poor material that about one-third of it was entirely removed to better bottom about 10 ft. above the water surface and the balance was notched out parallel to the deck and faced with 2 ft. of concrete which formed an extension of the deck and tied in with the cut-off wall at the base and a low section of dam on the top of this portion of the island.

The total length of the spillway or overflow section extending from the left abutment to the non-overflow section on the island is 404 ft. 10 in. and has a rounded crest and heavy bucket and

apron extending over the downstream side of the island far enough to fully protect this portion of the structure from undermining. The buttresses in this portion are pierced near the top with openings used for a runway extending entirely through the dam from the power house up through the left abutment.

The non-overflow or bulkhead section in which the buttresses extend 15 ft. higher and are covered only on the upstream side and top, is open on the downstream side. One bay in this

FIG. 7.—METHOD OF CONSTRUCTION AND FORM WORK.

section contains a large concrete fishway tank discharging into a concrete fishway.

Figs. 6 and 7 show the dam in course of construction and the various types of forms are illustrated in Figs. 8, 9 and 10.

Because of the necessity of providing adequate trash rack area, supporting penstocks and meeting the requirements imposed by the high tension transformers and wiring installed here, the power-house section of the dam is quite complicated and required considerable thought and skill to work out the situation satisfactorily.

To locate the penstocks at convenient distances and properly support them on the adjoining buttresses as they pass from the intakes through the dam to the turbines, the spacing between buttresses is here made 14 ft. instead of 18 ft. as in the balance of the construction. An extra thickness of deck with additional reinforcement is provided around the penstock intakes and to carry the weight and prevent vibration several extra heavy struts are built against each penstock so as to tie across to the adjoining buttresses.

To provide means for getting machinery into the power house, which is built against the downstream ends of the buttresses of the power-house section, a railway connection is

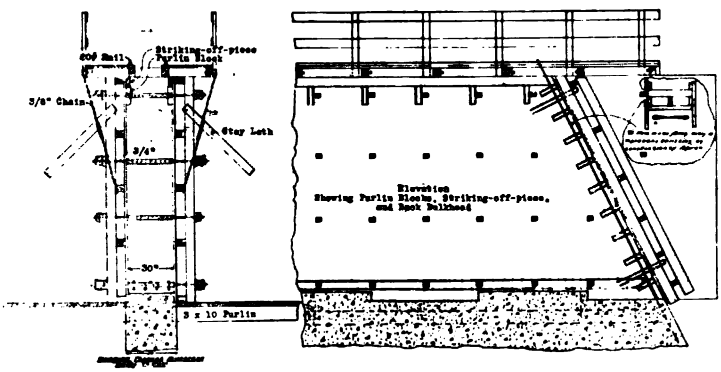

FIG. 8.—METHOD OF HOLDING TONGUE AND HAUNCH FORMS IN PLACE.

extended along the top of the right end abutment so as to enter directly on the unloading platform where crane service is available to quickly lower material to the power-house floor level and where, after being transferred by a suitable truck, it is handled by the power-house crane.

The deck for the intake section of the dam is set back from the face of the deck below it to allow space behind the trash racks for head gates and stop logs and to permit easy access of the water from racks to penstock opening and is of special construction on this account.

The racks, hoists and loading platform are housed by a concrete structure extending the full length of the power house. The

power house has a solid concrete substructure with flood protection for a height of 14 ft. above which is a concrete and steel superstructure.

The initial installation consists of three 6000 h.p. 240 r.p.m. twin flume, center discharge, volute casing turbines, of special design originally suggested by us, each connected to a 3300 k.w., 3-phase, 10,000-volt generator, and penstock and power-house space is provided for two more units.

These turbines feed from below through a cast iron Y pipe connection to their penstocks, have substantial cast iron bed plates and casings, presenting a neat and compact appearance.

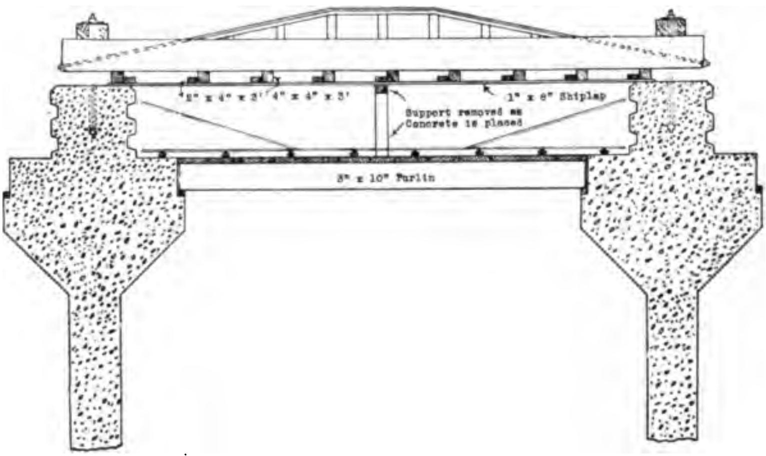

FIG. 9.—METHOD OF ANCHORING DECK SLAB FORMS.

The oil type governors are provided with the usual distant control and other auxiliaries as well as extra flyball head for emergency "shut downs" in case the regular head fails to act.

The generators are so supported on the foundations as to have ample ventilation in all parts. A direct connected exciter on the outboard end of each shaft and one reserve combined turbine and motor-driven exciter provide the necessary exciting current.

The 3-phase, 60-cycle, 33,000-volt delta and 57,000-volt Y step-up transformers, each carrying the load of the adjacent generator, are located in the dam back of the turbines and imme-

diately above them are the 57,000-volt high tension and the 10,000-volt low tension oil switches, bus bars, electrolytic lightning arresters, etc., connecting with the two 3-phase transmission lines to Portland.

CONSTRUCTION.

The scarcity of suitable construction materials was at first a matter of considerable importance but with the assistance of Mr. Robert S. Edwards, consulting chemical engineer, of Portland, who was retained to investigate the quality and costs of

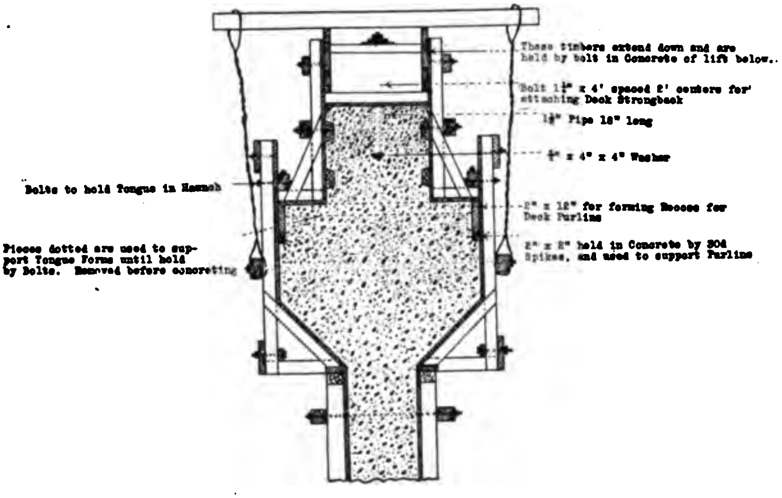

FIG. 10.—FORMS FOR BUTTRESS.

sand, stone and cement available, satisfactory materials were obtained.

Sand for construction in this section is ordinarily pumped from the Columbia River, but on investigation of the possibilities along the railway right of way a large pit was discovered which provided sufficient sand and gravel of good quality at considerable saving for not only this job but for much of the other construction work which the company had on hand.

The only rock quarry opened up was also along the railway right of way, and though far from satisfactory produced a good quality of basalt rock at reasonable cost.

Because of the fact that the cement manufacturing business on the Pacific Coast was still in a rather non-systematized state both as regards the standard of uniform quality and the matter of regular supply, the purchase of cement was considered only after a careful personal investigation and report on the available mills on the coast to determine the history and quality of the product and the probability of securing satisfactory shipments. Considerable time and expense were devoted to this investigation, but with these data in hand an advantageous contract based on satisfactory specifications was closed for sufficient cement to meet the company's requirements for several years at a considerable saving.

The time for construction being quite limited with a large amount of excavation to be made, the construction plant investment was considerable. The equipment for excavation consisted of 8 steam drills assisted by 5 derricks, 1 electric locomotive and a steam locomotive crane. For the core drilling and grouting there were required 6 Davis-Calyx core drills, pressure tanks for water testing and 2 motor-driven Peerless air compressors and 2 Canniff air-stirring tanks for grout supply.

The sand pit was equipped with screen for eliminating the large stone and coarse gravel and with several bins which were arranged to empty directly into the railway cars which passed beneath them. The quarry equipment included a motor-driven crusher and screens together with bins for dumping directly into the cars. At the site liberal sand and stone bins were provided for each of the 2 electrically driven 1-yd. mixers first installed. A third steam-driven mixer was later installed and the three together were utilized to supply the two cableways which carried the concrete directly onto the work.

Cement and lime storage sheds were also provided close to the mixers to meet the ordinary needs between receipt of shipments.

The concrete mixing plant and material storage bins and sheds were located beyond the right end of the dam where they were fed by electric railway supply trains from the company's line to Cazadero. The mixed concrete was then fed to the cableway buckets for distribution over the work. On the buttress forms tracks were arranged to carry a suitable concrete car which

received the concrete from the cableway bucket and distributed it as required in the buttress.

The construction necessarily proceeded along lines dictated to a considerable extent by the large excavation necessary on the island and the time required for installation of machinery in the power house.

Starting on the left channel during the beginning of the dry season the cut-off wall was installed, holes drilled and grouted and the first lift of a buttress constructed in midstream to serve in dividing this channel during the following dry season when completing the construction. During this time excavation and grouting were proceeding on the left bank and island above water line to permit concreting at these points to follow as soon as possible. Next the right or power-house channel was permanently unwatered and the old river chasm found below the bed of the stream was cleaned out and work was pushed on grouting and concreting to permit installation of racks, penstocks and machinery in advance of the completion of the spillway. The great amount of excavation on the island hindered this portion of the work to some extent, but fortunately no unusual floods were experienced and the entire structure up to the left river channel was completed early enough so that with the installation of two special openings in addition to the four sluice gates the stream flow could be passed until such time as it might be possible to discharge it over the crest.

The construction work started in June, 1910, and the water first passed over the spillway November 7, 1911, which in view of the conditions to be met may well be considered quite rapid construction. The largest average force employed for any month was 655 men. The record excavation was 9476 cu. yd. per month. The greatest yardage of concrete placed any month was 8325 cu. yds.

Credit is due Mr. Robert S. Edwards, who had charge of the cement investigations and testing, and Mr. Frank R. Fisher, resident engineer on the later part of the construction, for data used in this paper.

UNIT COSTS OF REINFORCED CONCRETE FOR INDUSTRIAL BUILDINGS.

By Chester S. Allen.*

Unit costs are harmless when used with judgment and prudence, but likely to bring remorse and anguish when employed promiscuously. Rare and talented indeed is the man who possesses the experience, judgment and intuitive sense to know when, where and how to properly modify any tables or statements of unit costs to meet the peculiar conditions of each individual case. While the figures given in this paper are all taken from structures erected during the past two years under the writer's supervision, the wide range of territory, local conditions, and different seasons of the year under which the various pieces of work have been executed are so great as to render the information of value only in a very general way.

As a general proposition it has been found that reinforced concrete is the lowest-priced fireproof material suitable for factory construction and while it is true that its first cost will generally run from 5 to 20 per cent higher than first-class mill construction, recently in several instances, with lumber at a high price, reinforced concrete has worked out cheaper than brick and timber. It is especially adapted to heavy construction and for heavy loads of 200 lb. per sq. ft. and over where the spans are 18 to 20 ft. centers, not even timber can compete with it.

The unit costs of projected or completed buildings are commonly figured either as so much per cubic foot or as so much per square foot of area occupied. Table I gives the unit costs both on the sq. ft. and the cu. ft. basis, together with a general description of a number of reinforced concrete industrial buildings of different types erected during the past two years. It will be seen from an examination of this table that the average cost per sq. ft. of these buildings, excluding the one-story structures, was $1.12; while the average cost per cu. ft. was 8.7 cts. The one-story structures both had reinforced concrete sawtooth roofs

* Engineer, Lockwood, Greene & Company, Boston, Mass.

TABLE I.—UNIT COSTS OF REINFORCED CONCRETE INDUSTRIAL BUILDINGS.

Type of Building.	Dimensions in feet.	Number of Stories.	Story Height, ft.	Live Load, lb. per sq. ft.	Type of Construction.	Column Spacing.	Total Cost in Dollars.	
							Per sq. ft.	Per cu. ft.
Machine Shop..................	126 x 50	4	12	150	Beam	10 x 24	1.17	0.09
Cotton Mill....................	550 x 129	2	16	75	Beam	10.8 x 25	.98	0.07
Factory.........................	400 x 52	4	12.5	150	Flat Slab	17 x 20	1.09	0.077
Weaving Mill...................	140 x 60	5	12.5	150	Flat Slab	17.6 x 20	1.50	0.12
Knitting Mill...................	220 x 75	2	14	125	Beam and Girder	22 x 25	1.09	0.073
Cartridge Factory..............	223 x 56	2	16	300 and 1000	Beam and Girder	18.6 x 18.6	1.55	0.10
Weave Shed.....................	341 x 231	1	....	125	Sawtooth Skylights	13 x 21.4	1.79	0.07
Machine Shop..................	220 x 100	1	....	....	Sawtooth Skylights	20 x 20	1.75	0.10
Store House....................	181 x 56	4	14.5	150	Flat Slab	18 x 20	1.15	0.07
Store House....................	580 x 109	10	12	250	Beam and Girder	$19.3 \times \begin{Bmatrix} 19 \\ 25 \end{Bmatrix}$	0.85	0.071
Store House....................	580 x 244	4	15	350	Flat Slab	$\begin{Bmatrix} 18.6 \\ 25 \end{Bmatrix} \times 25$	0.76	.05
Store House....................	256 x 100	12	8	150	Flat Slab	16 x 16.8	1.04	.12

and the average cost per sq. ft. was \$1.77, while 8.5 cts. was the average cost per cu. ft. The above costs are for the finished buildings, including plumbing, but do not embody heating, lighting, elevators, sprinklers and power equipment. The cost per sq. ft. of floor area was obtained by dividing the cost of the building by the total number of sq. ft. of floor area exclusive of roof area but including basement floors; and the cost per cu. ft., by dividing the cubical contents into the cost of the structure.

While no coal pockets are included in Table I, it has been our experience that above 3000 tons capacity reinforced concrete elevator coal pockets cost from \$5.50 to \$7.50 per ton of capacity. Standpipes, exclusive of the foundations, average from 2½ to 3 cts. per gallon of capacity.

On much of the reinforced concrete work which has been done under our supervision it has been possible, owing to the contract being either on a percentage or cost plus a fixed sum basis, to obtain quite accurate and comprehensive cost data. This data, of course, is only of particular value when all the local color of each specific case is known, but the average results are at least interesting.

The average unit cost of the 1-2-4 concrete in the floors including the beams, girders and slabs, was \$6.10 per cu. yd., and for the columns \$6.70 per cu. yd. Where 1–1½–3 mixture was used for the columns the average cost was \$7.60 per cu. yd. This cost was made up of the items of cement, sand, stone or gravel, labor and plant. The cement of course varied greatly with the demand, but the average net cost was \$1.35 per barrel including 3 cts. for tests. The sand averaged 80 cts. per cu. yd. and the crushed stone \$1.25 per cu. yd. The cost of labor of unloading the materials and mixing and placing the concrete varied from 65 cts. to \$2.90 per cu. yd. The cost of plant, consisting of freight, depreciation or rental of mixing and hoisting towers, erection of same, power and coal, and losses and waste on the small tools, ranged from 50 cts. to \$1.50 per cu. yd. of concrete placed.

Next to the proper design of the structural features of a concrete building, the economical design of the form work is of paramount importance. The truth of this statement is borne out by the fact that on the average job the cost of the forms

amounts to about one-third the cost of the entire structure. On the buildings under consideration the average cost of the forms for the floors, including beams, girders and slabs, was 10 cts. per sq. ft., and for the columns 13 cts. per sq. ft. The lowest cost was in a building of the girderless or flat slab type of construction, where by the intelligent use of corrugated iron for the slab forms the cost of the floor forms, including wall beams, was 7 cts. per sq. ft. The highest cost was for an artistic but not elaborate overhanging cornice on a 12-story building, and was 32 cts. per sq. ft. This last item rather forcibly demonstrates that any attempt at architectural development is very apt to be a costly proposition.

The cost of the labor of making, erecting and stripping the forms varied according to the price of lumber, design of the structure, method of forming, character of the supervision and the skill of the workmen from $4\frac{3}{4}$ to 12 cts. per sq. ft. The cost of lumber, nails and oil divided by the sq. ft. of forms averaged from $2\frac{1}{4}$ to $4\frac{1}{2}$ cts. per sq. ft.

The cost of bending and placing the reinforcing metal, including the necessary wire, averages $10 per ton, the range being from $5.75 to $17.20 per ton.

Granolithic floor finish $1\frac{1}{4}$-in. thick when laid before the concrete below it had set so as to form one homogeneous slab, cost on the average of $4\frac{1}{2}$ cts. per sq. ft. When put on after the rough concrete slab, the cost averaged 7 cts. per sq. ft.

Inasmuch as the only economical design of a reinforced concrete structure is one which closely resembles that of the steel skeleton type, the relative cost of the various materials commonly used for curtain walls under the windows may be of interest. The writer has used brick, vitrified tile, concrete blocks, cast concrete slabs and solid concrete walls for this purpose.

The most common type of curtain wall has been either an 8-in. or 12-in. brick wall resting on the concrete wall beam. The average cost of these walls has been 45 cts. per sq. ft. There is practically no difference in cost between the 8-in. and the 12-in. brick curtain wall, as the saving in material is offset by the great amount of extra labor in culling and laying the thinner wall.

An excellent and inexpensive curtain wall is constructed

by using 8 x 12 x 18 in. vitrified tile. This is a non-absorbent wall and when properly laid in cement mortar makes a tight weather-proof curtain wall. The cost of this wall averages about 25 cts. per sq. ft. If the tile is plastered both sides, the cost is about 38 cts. per sq. ft.

Where 8-in. concrete curtain walls were cast in place after the skeleton frame was completed, the average cost was 40 cts. per sq. ft., and when poured simultaneously with the columns 48 cts. per sq. ft. 4-in. cast concrete slabs cost about 35 cts. per sq. ft.

While concrete blocks make a very cheap and light curtain wall, the price being about the same as for the 8-in. tile, the writer's experience with them has been rather unfortunate on account of the extreme porosity of the blocks used.

Where the location of the buildings has demanded special treatment of the exposed surfaces, they have generally been specified to be rubbed with a block of carborundum. The average cost of this work has been 4 cts. per sq. ft. In two instances portions of the structures have been bush hammered with a resulting average cost of 7 cts. per sq. ft.

Concrete piles were used on the foundations of several of the buildings and the average cost of the piles was $1.15 per lin. ft.

The most common methods of waterproofing concrete structures are by the introduction of foreign ingredients into the concrete, by the application of a compound to the concrete surface, by the use of paper or felt waterproofing, and by accurately grading and proportioning the aggregates and the cement.

Where an addition of hydrated lime in the proportions of 10 per cent to the weight of the cement has been used, the added cost to a cubic yard of 1–2–4 concrete has been 50 cts. Patented compounds have cost from 25 to 35 cts. per sq. ft. of surface covered. On horizontal or inclined surfaces, we have sometimes used a granolithic surface of rich mortar of Portland cement and sand or Portland cement and screenings in the proportions of 1–1, laid at the same time as the base and troweled as in sidewalk construction. The cost of this work has been about 5 cts. per sq. ft.

Taken as a whole, the lowest possible cost on a reinforced

concrete building can be obtained only by a careful study of each particular case to determine the cheapest type of construction and most economical spacing of columns. As a general proposition it has been found that for light loads with ordinary beam and girder construction the most economical spacing of columns is 18 ft. each way and for flat slab construction 20 ft. each way. For heavy loads such as 300 lb. per sq. ft. and over, it has been our experience that the cheapest column spacing for beam and girder construction is 15 ft. by 15 ft., and for flat slab construction 17 ft. by 17 ft. In arriving at the most economical layout it is always well to bear in mind that the construction which allows the greatest simplicity of form units, together with the maximum number of repetitions of same, is invariably the one that will work out cheapest in the end. The fact that the actual amount of concrete or reinforcement required for a certain floor construction is less than that required in another by no means implies that this is actually the cheapest floor construction, as the unit labor of the form work may easily have been increased out of all proportion.

REINFORCED CONCRETE CONVENTION HALL AT BRESLAU, GERMANY.*

By Dr. S. J. Trauer.†

There is being built at the present time in the City of Breslau, Germany, a large convention and exhibition hall (Fig. 1), surmounted by the largest concrete dome in the world. The building, of reinforced concrete throughout, has a seating capacity of 9000 persons and standing room for 12,000 people.

The structure (Fig. 2) consists essentially of a main hall,

FIG. 1.—CONVENTION HALL, BRESLAU, GERMANY.

circular in form, connecting directly with four semi-circular halls called Apsiden, all of which are surrounded by a lower circular hall serving exhibition purposes. A dome of 213 ft. span with a rise of 65 ft. rests on the substructure, 65 ft. in height, and carries a light dome.

The substructure (Fig. 3) consists of four main arches, (*A*) of 131 ft. span and 62 ft. rise, circular in plan and supported by four piers. Each main arch (*A*) is supported from the outside by four auxiliary arches (*B*) which cover a smaller semi-

*Translated from author's notes by the Secretary.

†City Bridge Engineer, Breslau, Germany.

circular hall called an Apside. These auxiliary arches rest on individual piers and take the outward thrust from the main arches. The area of the cross-section of the main arch at the skew-back is 24 sq. ft. and at the crown 3.6 sq. ft., all reinforced with round bars of 1.18 in. diameter. The main arches are solid and are subject to their own temperature stresses only. The

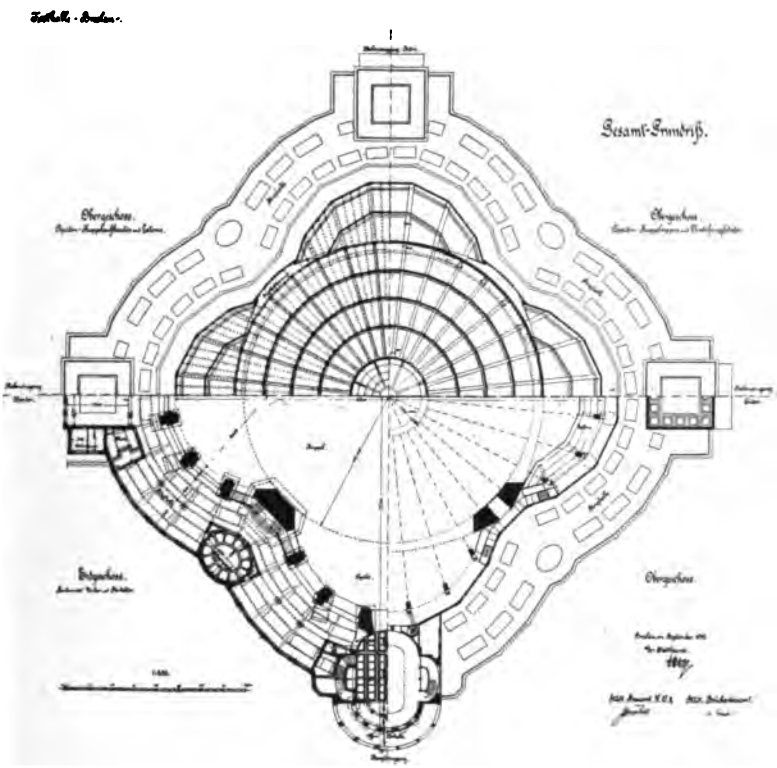

FIG. 2.—PLAN OF CONVENTION HALL, BRESLAU, GERMANY.

horizontal thrust amounts to 771 tons, the pressure at the skew-back is 1323 tons and in addition the main arches are subjected to torsion and bending.

The auxiliary arches (*B*) which receive about 220 tons thrust from the main arches, are connected with the main arches and the small abutments through ball bearings (*C*) of malleable steel

castings so that the auxiliary arches receive only axial stress. These arches are 3.28 ft. in width and 5.28 ft. in thickness.

The substructure is entirely independent of the dome itself and supports the latter by means of steel roller bearings (*D*) under each rib, the bearings having radial movement. So that in general only vertical stresses are transmitted to the substructure, which is, therefore, not subject to the temperature stresses in the dome. The wind pressure on the dome is not transmitted in a

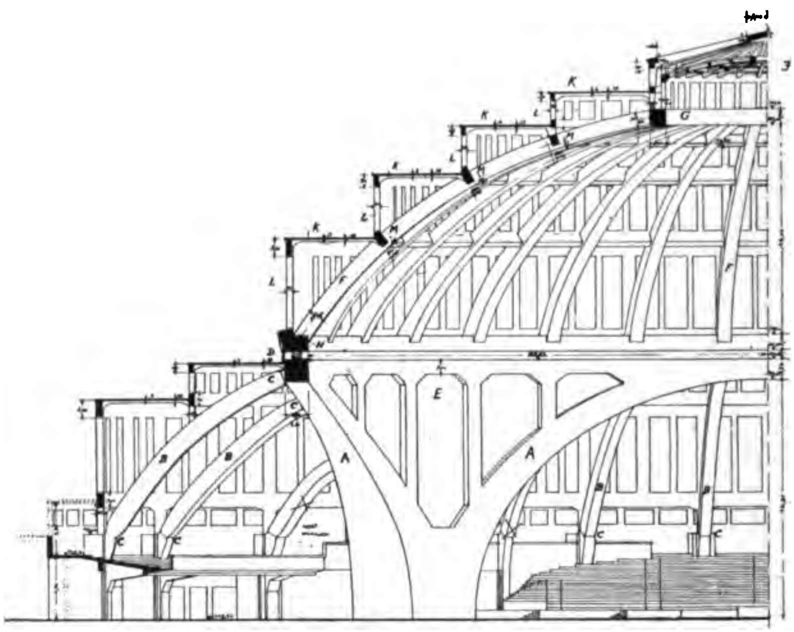

FIG. 3.—SECTION THROUGH MAIN AXIS.

radial but in a tangential direction, that is, the direction in which the four main arches have the highest power of resistance. The frames (*E*) over the four abutments of the structures are calculated to take up the total wind load on the dome.

The dome consists of 32 half ribs (*F*), which bear on the top against the pressure ring (*G*) and on the bottom against the tension ring (*H*). The pressure ring of 47.2 ft. inside diameter is surmounted by an upper light dome (*J*). The pressure ring with a cross-sectional area of 19.7 sq. ft. carries 551 tons in com-

FIG. 4.—RIVETED STEEL TENSION RING AND HAUNCH OF ONE RIB.

FIG. 5.—SUBSTRUCTURE AND CENTERING FOR HALF-RIBS OF DOME.

FIG. 6.—CONCRETING HALF-RIBS OF DOME.

FIG. 7.—VIEW OF INTERIOR.

pression and the tension ring (Fig. 4), with a cross-sectional area of 29.9 sq. ft., carries 551 tons in tension. The ribs carry a horizontal thrust of 110 tons and 115 tons on the skew-back and have a cross-sectional area which increases from 3.4 ft. x 2.13 ft. to 3.94 ft. x 2.62 ft. The ribs and the tension and pressure rings are reinforced with round rods. The ribs are strengthened by means of rings (*M*) and horizontal circular slabs (*K*). The tension ring is of riveted steel construction.

The roof supports are almost horizontal, the windows (*M*) are vertical so that in the case of snow the interior illumination will not be affected.

The architectural design was made by the City Building Engineer, Berg; the calculations by the City Bridge Engineer, Dr. Trauer, and the verification and construction by the firm of Dyckerhoff & Widmann, of Dresden, Germany.

THE SUITABILITY OF CONCRETE FOR GAS HOLDER TANKS.

By Herbert W. Alrich.*

In its purpose the gas holder represents the common feature of all manufacturing enterprises—it is the receptacle for the storage of the product. But in their design, gas holders are not resembled by any structure or equipment employed in any other line of industry. A prominent engineering writer has recently said that the modern gas holder is "A magnificent achievement in engineering, and one of the wonders of it is the telescopic feature." As the majority of engineers are not familiar with the mechanical features of a gas holder, a description is given; for in discussing tank design, it is necessary to consider the structure as a whole.

There are three principle parts to a gas holder; the tank, the telescopic sections or gas holder proper, and the guide frame. Each of these parts differs from the others in function and in the type of construction. The tank, resting upon the ground, is filled with water up to a level about 15 in. below the top. The gas holder proper consists of the telescopic sections, of which there are five in the case of the largest holders that have been built in this country. These sections consist of cylindrical steel shells, concentrically located with relation to each other and to the tank. When the holder contains no gas, these shells are nested together, resting upon the bottom of the tank and submerged almost completely in the water which the tank contains. (See Fig. 1.) The outer-most shell is usually about 3 ft. less in diameter than the tank, and each succeeding section is about 2 ft. 9 in. less in diameter than the preceding one. All of the sections are open at each end, with the exception of the innermost one, the upper end of which is enclosed by crown plating, having a spherical form.

The inlet and outlet pipe connections enter the tank through

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the bottom and pass vertically upward through the water and terminate at an elevation above the water and just under the crown of the inner section. When gas is admitted to the holder, it first acts upon the crown, lifting that section gradually out of the water as shown by Fig. 2, until the cup, constructed around the lower edge of that shell, engages the upper edge of the next

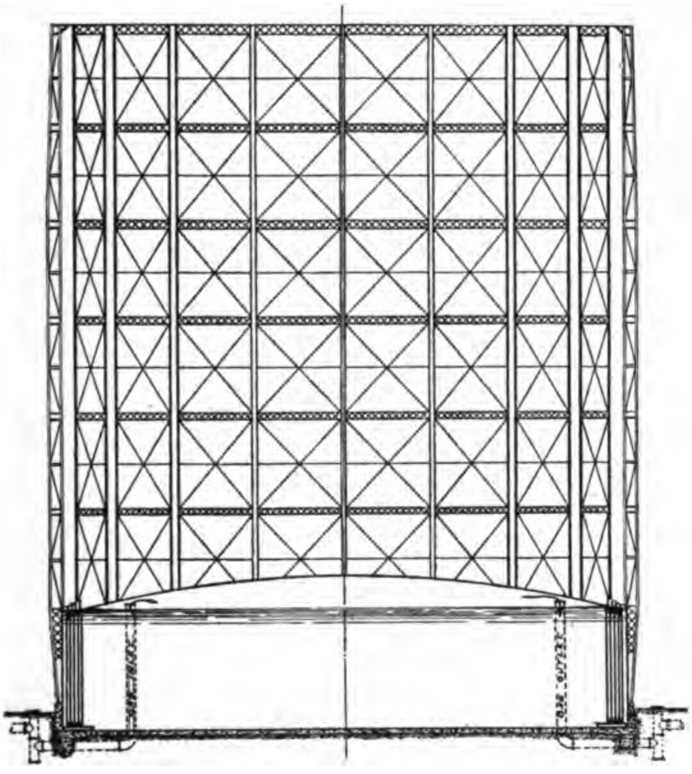

FIG. 1.—CROSS SECTION OF A GAS HOLDER GROUNDED.

outside section. To accomplish this engagement, the upper edge of each outer section is constructed in the form of a continuous annular hook, called the grip, and correspondingly the lower edge of each section, except the outermost one is formed into a continuous annular cup. Both the grip and the cup are identical in construction, and differ only in the respect that the grip is inverted to permit interlocking with the cup as shown by Fig. 3.

As the inflation of the holder continues, each succeeding section is lifted out of the water, and, in turn, picks up the next outer section. As each interlocked cup and grip pass upward out of the tank, there is carried along that quantity of water which is necessary for forming a hydraulic seal against the maximum

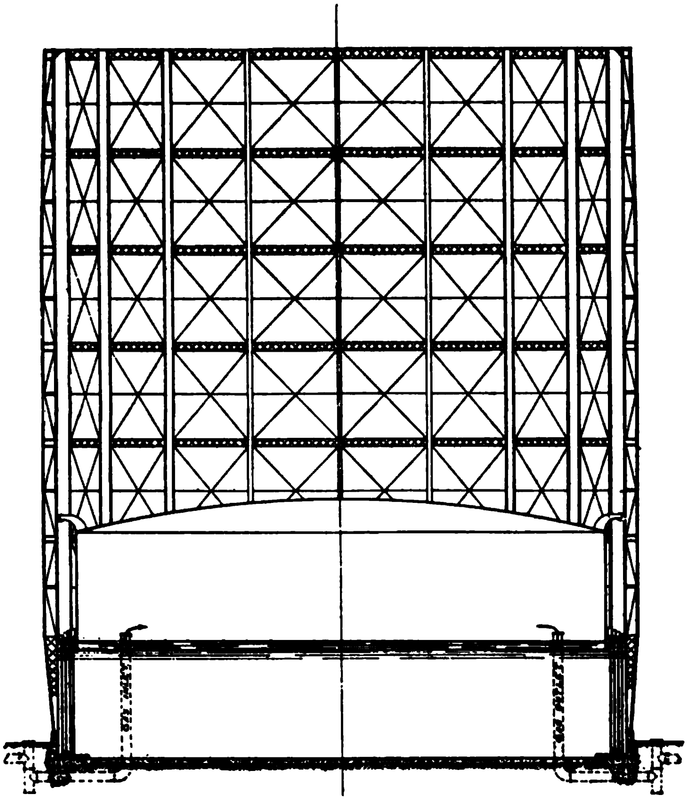

FIG. 2.—HOLDER PROPER JUST BEFORE ENGAGING AN ADDITIONAL SECTION.

pressure of the holder. This action continues until the holder is filled to the limit of its capacity as shown by Fig. 4.

While thus ascending, or conversely descending, the sections are maintained in their relative concentric positions by two sets of rollers. First, the rollers spaced equi-distant around the lower edges of the shells, and second, the rollers mounted on brackets attached to the upper edges of the shells. The first

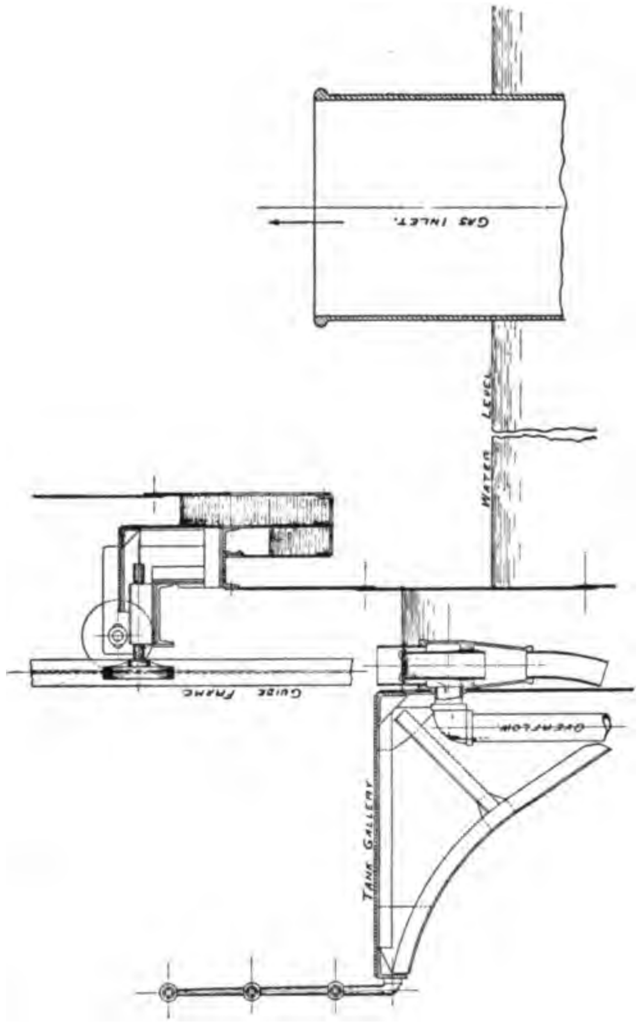

FIG. 3.—CROSS SECTION SHOWING THE HYDRAULIC SEAL BETWEEN TWO OUTER SECTIONS.

set are known as the internal rollers, and travel upon vertical guides, located on the inner surface of the tank and also on the inner surface of each shell, except that of the innermost section. The other set of rollers travel upon the guide rails, which are carried by the guide frame. The guide frame consists of vertical columns spaced equi-distant, and about 30 ft. apart around the

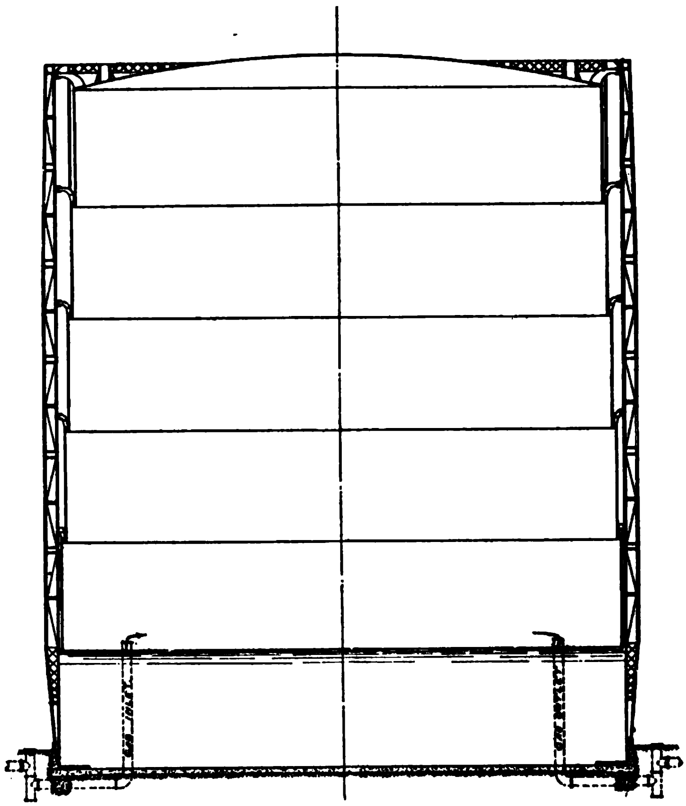

FIG. 4.—HOLDER FULLY INFLATED.

circumference of the tank. These columns are connected by horizontal and diagonal cross bracing. The frame is carried to such a height as will provide guidance for the inner section in its maximum upward travel. It will be apparent from this discussion that the enormous pressure of the wind against the inflated sections is ultimately transmitted to the guide frame by the tank.

In the selection of materials in manufacturing, we are subject to an inexorable law of suitability. In producing any manufacture, there will be ultimately employed that material which possesses preponderant advantage. The use of any other cannot long be sustained. From the very beginning of the gas industry, 100 years ago, the guide frames and holder sections have been built exclusively of iron or steel. It does not appear that any attempt was ever made to use any other materials. In the construction of the tanks, however, many different materials and combinations of materials have been employed. It is true, however, that in this country, until recent years, all holder tanks were built either of brick or stone; but in England, the birthplace of the gas industry, there were constructed tanks of almost every conceivable type, many of the designs being more fanciful than practical. It was inevitable that those constructions without merit should disappear until the prevailing practice, throughout the world, during the last twenty years, may be stated as limited to steel tanks placed upon the ground, and brick or concrete tanks located below the ground. During the last ten years the preponderance of advantage in this country has been in favor of steel tanks for all holders, large or small. In England, though concrete tanks had been constructed as far back as the year 1870, the practice of the last decade has been quite uniformly to build small tanks of steel, and large tanks of brick, there being but few instances of concrete construction. In Continental Europe, practice for a decade has followed the English, with the exception that during the last three or four years there has been a marked tendency toward the exclusive employment of steel, though a few concrete tanks of small size have also been constructed.

The competitive, economic and geological conditions existing in England, all favor the brick tank for large holders. While the population of the Island of Great Britain is somewhat less than that of the United States, the number of concerns engaged in holder construction is very much greater, in fact the building of the largest holders in this country is confined to a very few companies, having special qualifications and known responsibility. Hence, the commercial conditions prevailing in this country have enabled these firms to develop extensive equipments, which have

wonderfully facilitated the construction of steel tanks. When selecting sites for large holders in England, they appear to have generally encountered formations of clay possessing remarkable stability. This has permitted them to construct economical tanks by lining a cylindrical excavation in the clay with brick walls, which are thinner than our geological conditions will generally permit. Unskilled labor is relatively very cheap in England, and in some cases, the cost of construction has been partly defrayed by the commercial value of the excavated clay In the case of the 10,000,000-cu. ft. holder erected at Manchester in 1909 and 1910, the brick for the tank walls were manufactured upon the site from the excavated clay. It is a fact, however, that the time required for the construction of this particular holder would be regarded as economically impossible in the United States. The result of all of these conditions has been that the English holder builders have not developed the special equipment required for the construction of very large steel tanks, nor does it appear that the British steel plants produce plates suitable for the purpose.

To confine the discussion now to American conditions the controlling requirements in the design of a gas holder are:

1. Structural Stability.
2. Economy.
3. Rapidity of Construction.
4. Durability.

Certain physical phenomena having to do with the distribution of gas, require the holders to be located upon the lowest available ground. In this country such sites rarely consist of a very stable geological formation, and are frequently reclaimed land. Hence, if under the usual conditions there be undertaken the construction of an underground masonry tank, there will probably be encountered such difficulties as to greatly enhance the cost of the work and prolong the period of construction. If it should be suggested that a concrete tank might be built above the ground, a moment's reflection upon its relation to the guide frame is sufficient to dispose of the proposition.

During the last eight years the writer has participated in the design and construction of the three largest concrete holder tanks in the world. Two of these tanks are 300 ft. in diameter by 48

ft. 3 in. deep, while the third is 189 ft. in diameter and 41 ft. 6 in. deep, all inside dimensions. In referring to these particular tanks, and also throughout the discussion, he wishes to be understood as speaking for himself alone. The work on these three tanks proceeded in such simultaneous relation as permitted any improvement in methods developed on one, to be employed to advantage on another.

At the time the tanks were being designed, Mr. William H. Bradley, the Chief Engineer of the Consolidated Gas Company of New York, made a thorough inquiry as to the design of any other concrete tanks that had been constructed anywhere in the world. There was found little precedence to consider. As regards the two 300-ft. tanks, the undertaking was one of unusual magnitude, for the holders were the largest that have ever been built. The possibilities of nickel steel were not at that time fully understood, hence the construction of steel tanks would have been unthinkable, as there would have been required plates 4 in. thick, connected by rivets 3 in. in diameter. It was evident that masonry construction of some kind must be employed. A number of different designs were worked out, analyzed and compared, resulting in the conclusion that the most advantageous construction would be a plain annular wall of reinforced concrete, and all three tanks were thus built. The work upon all three tanks was accomplished with entire success. While there were special conditions, the controlling factors in these instances, and determining the selection of concrete construction, the writer concludes that steel is far preferable, as a general proposition. The smallest of these three tanks required a full year for its construction. An entire holder of the same size having a steel tank, can be constructed in seven months, without special effort. It has also been demonstrated to the satisfaction of the writer, that in no case where a steel tank is at all possible, will the cost reach 70 per cent of the outlay required for a corresponding concrete construction.

The question as to whether steel or concrete tanks are the more durable, must be settled in another generation; but the writer believes that the steel tanks are the more expedient. He has knowledge of several brick tanks having been badly damaged by blasting in their vicinity. From his knowledge of other con-

crete structures having been badly cracked in a similar way, he must conclude that concrete tanks are not immune from the hazard. The durability of concrete tanks is also open to question, in view of the peculiar behavior of concrete observed by the writer, in the instance of two of the most prominent engineering undertakings in the United States, in which cases, several years after the concrete had set, masses of it became soft and pulpy, requiring replacement. A holder tank is not such a structure as one upon which such repairs can be easily made, but it is one of such importance as to forbid speculative constructions.

The writer has examined and reported upon a large number of steel tanks, some of which had been in service for over twenty years. While these older tanks are doubtless constructed of a steel far inferior to that which is now produced, all of these tanks, with one exception, could survive the youngest man now engaged in the gas business. It has sometimes been urged against the durability of steel tanks, that all favorable conclusions had been formed from exterior examinations. The writer has had the opportunity of examining internally, some of the oldest steel tanks in existence, through which he concludes that, excepting at the water line, there is no perceptible deterioration. In one case, the holder had been employed for twelve years, in connection with a process of gas manufacture now obsolete. As a part of this process, a non-luminous gas containing some sulphur was stored in this holder, which resulted in the sulphur being absorbed by the water in the tank. The writer found the interior of this tank to be but very slightly pitted.

The writer would state it as his conviction, that while not possessing one single element of advantage over steel tanks, that concrete tanks are subject to the following comparative disadvantages:

1. Increased cost, generally 75 per cent.

2. Longer period required for construction, generally 100 per cent for the entire holder.

3. Liability to impairment from unforeseen or unavoidable causes, such as internal stresses, bad water, oil or alkaline soil and blasting.

4. Difficulty of making repairs and obtaining the original strength in case of rupture.

5. Possibility of corrosion of reinforcement, the vital element of the tank's strength.

6. Inaccessibility of metal to inspection.

7. Greater load on foundations.

8. Requires the most rigid inspection and supervision to control the quality of the work during construction.

9. Liability to serious difficulties from storms during construction, due to the depth of excavation, the requirement of maintaining adjacent streets, requiring sheet piling 50 ft. deep and massive shoring timbers. Also liability of damage to the concrete work itself from the same cause.

In submitting a discussion of this question to the American Gas Institute in 1910, the writer urged the following as the twelve distinct advantages of steel tanks when compared with those of masonry:

1. Less cost.

2. Shorter period required for construction.

3. The ease with which the quality of the work may be controlled during construction.

4. The high state of development in fabrication and erection.

5. Susceptibility to exact computation and greater reliability under stress.

6. Accessibility for inspection.

7. Tank may be placed at any elevation with relation to the ground line that may be desired.

8. No liability to damage by storm during construction.

9. No internal stresses from shrinkage or temperature that are serious.

10. Possibility of rectifying an unequal settlement.

11. No liability to cracking from undetermined causes.

12. The ease of making repairs and obtaining the original strength.

Every one of the enumerated disadvantages of concrete tanks, and advantages of steel tanks might be separately extended and elaborated, but the writer will confine himself to items 2 and 4 under steel tanks.

The bottom of a steel tank, excepting an outer course, consists of rectangular steel plates, generally about $\frac{3}{8}$ in. thick, con-

nected by single riveted lap joints. The outer course consists of heavy segmental plates, which conform the bottom to the circle. Riveted around the outer edge of this outer course, is the bottom curb, which, in the case of a large tank, will be an 8 x 8 x $1\frac{1}{8}$ in. angle. The bottom course of curved plates, forming the shell of the tank, is riveted to the upstanding leg of this curb angle. The next course of plating is then attached to the lowest course by a single riveted lap joint running horizontally around the tank circumference. The other courses are connected in a similar manner, the entire number in the height of the tank usually being about eight or nine. It is necessary for the vertical joints, which occur about 30 ft. apart around the circumference, to be spliced in a manner capable of resisting the full circumferential tension. To accomplish this, the joints are covered by double butt straps, each such splice being quadruple riveted with double shear rivets, and triple riveted with single shear rivets. The total number of rivets in the cylindrical shell of the tank alone, may be as many as 25,000.

After the bottom of the tank has been completed, a vertical steel post is erected at its geometric center. A circular rail is also laid concentrically upon the tank bottom. Mounted upon trunnions, upon the top of the central steel post, are two radial traveler arms, the outer ends of which are supported by legs having wheels running upon the circular rail. These radial travelers are equipped with such suitable attachments, as will permit one of them to be used for assembling the curved plates while the other supports the hydraulic-pneumatic riveting machine. Thus, as one traveler proceeds around the circumference lifting the plates into position, it is followed by the other with the riveter. Quite obviously, the first traveler is able to keep well ahead of the riveting, which permits that traveler to participate in the simultaneous erection of the five-holder shells. When the tank and the holder sections have been completed, the tall center post around which the travelers revolve, is replaced by a much shorter one, resting upon the center of the crown, while the outer leg of one of the travelers has its lower section removed, and thus shortened, it travels upon a circular rail which has been attached to the completed crown. The other radial traveler is entirely removed. After water has been placed in the

tank, the holder is gradually inflated with air, and as it rises the remaining radial traveler moves around the circumference, erecting the guide frame successively in tiers. It requires but little reflection to perceive that if the holder is to be provided with a concrete tank, any such co-ordination is impossible. As to the time required, it may be stated that in the instance of the largest steel tanks ever built, which were 251 ft. 3 in. diameter, the lower courses of which consisted of $2\frac{5}{16}$-in. plates and having 26 double butt joints in each course, the tanks were erected and riveted complete at the rate of 32 hours per course. One of these holders was entirely completed in $7\frac{1}{2}$ months. If there be a method of concrete tank construction which would have permitted the completion of that same holder in two years, it has not yet been found.

In closing, the writer will condense into a few words, his opinion on the matter as stated in another discussion. "He may be charged with entertaining a strong prejudice against masonry tanks. He will admit that he is opposed to any type of tank, 25 to 40 per cent of the cost of which may go into digging a hole in the ground instead of putting quality into the structure." To this he would add his conviction that a site which forbids a steel tank is not the place to put a holder.

PROTECTION OF STEEL IN CATSKILL AQUEDUCT PIPE SIPHONS.

By Alfred D. Flinn.*

Cement manufacturers and many cement users are already so familiar with the Catskill aqueduct which New York City is building to convey an additional supply of water from the Catskill mountains that a general description is not here necessary. This aqueduct is to have a nominal capacity of 500,000,000 gallons daily. In the 92 miles of its length, it crosses 14 minor valleys where metal pipe siphons were determined upon in preference to reinforced concrete pipes or deep pressure tunnels in rock; three of these siphons are west of the Hudson river, and eleven east; in seven of them the diameter of the steel shell is 9 ft. 6 in.; in four this diameter is 9 ft. 9 in., and in the remaining three, 11 ft. 3 in. These various diameters were determined by an economic distribution of the available fall or head. The length of the siphons varies from 608 to 6671 ft.; one of the 11-ft. siphons is 5584 ft. long; the total length of all siphons is 33,031 ft. The thicknesses of plates are $\frac{7}{16}$-, $\frac{1}{2}$-, $\frac{9}{16}$-, $\frac{11}{16}$- and $\frac{3}{4}$-in. The maximum heads on the siphons range from 50 to 340 ft. With one exception, each siphon rises to the hydraulic gradient at each end and is there connected by means of a concrete chamber to the adjacent portions of the cut-and-cover or tunnel aqueduct. They are all of open hearth steel. Ultimately there will be 3 pipes in each siphon, in order that the siphons may have the full capacity of the aqueduct and also provide for one pipe being temporarily out of service for cleaning, repairs or renewal. Only the middle pipe of each siphon is being laid at this time. The 7 more northerly siphons are included in Contract 62 and the 7 southerly siphons in Contract 68.

For years, both in connection with the work of the Board of Water Supply and in other engagements, several of the Board's engineers have been observing the results obtained by coating steel pipes with the asphalt, tar and other dips commonly em-

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ployed. Study had also been made of some special forms of protection of steel pipes. Excepting Portland cement grout or mortar used in one or two cases, in a small way, none of the coatings of which knowledge could be had gave evidence of real permanence, nor have they been fully satisfactory in other respects; therefore it was decided to jacket the steel pipes of the aqueduct outside with rich concrete, and to line them with Portland cement mortar. The outside concrete was to have a minimum thickness of 6 in., and the mortar lining a thickness of not less than 2 in.; after some experimentation, these dimensions were definitely adopted, as shown on Fig. 1, which is a reproduction of the drawing of the standard types of pipe construction.

Means for applying the concrete and mortar to the pipes so as to attain intimate, complete and permanent adhesion, in spite of the unavoidable distortions due to handling, temperature changes, and the water stresses, were studied, along with the most economical and feasible methods for the various steps in construction. Preparation of the steel was also carefully studied. Numerous experiments on a small scale, and finally on substantially full-size, were conducted. For the latter, a steel pipe 9 ft. in diameter and 12 ft. long, of $\frac{3}{8}$-in. riveted plates, was lined by plastering and by pouring grout or very thin mortar into the space between a cylindrical form and the inner surface of the pipe. In the plastering experiments several kinds of metal reinforcement were tried; also, terra-cotta and cement blocks or tiles were bedded on mortar and plastered. Briefly, it may be stated that no combination of plasterer's skill, with the various materials suggested, gave linings that were adequate and this method was expensive. The methods using solid tiles or blocks were more successful than those using any form of metal lath. But when removed, all the plaster coatings showed a tendency to separate at the surfaces between the successive layers. Grouting proved by far the most satisfactory and least expensive, and was adopted as the basis of the contracts.

In fabricating the pipes, the plates were bevel-planed on their edges, then punched for the rivets and then each plate was bent to proper radius by bending rolls. This bending cracked the mill scale and removed a considerable portion of it. Pickling was resorted to for the removal of the remaining mill scale, the

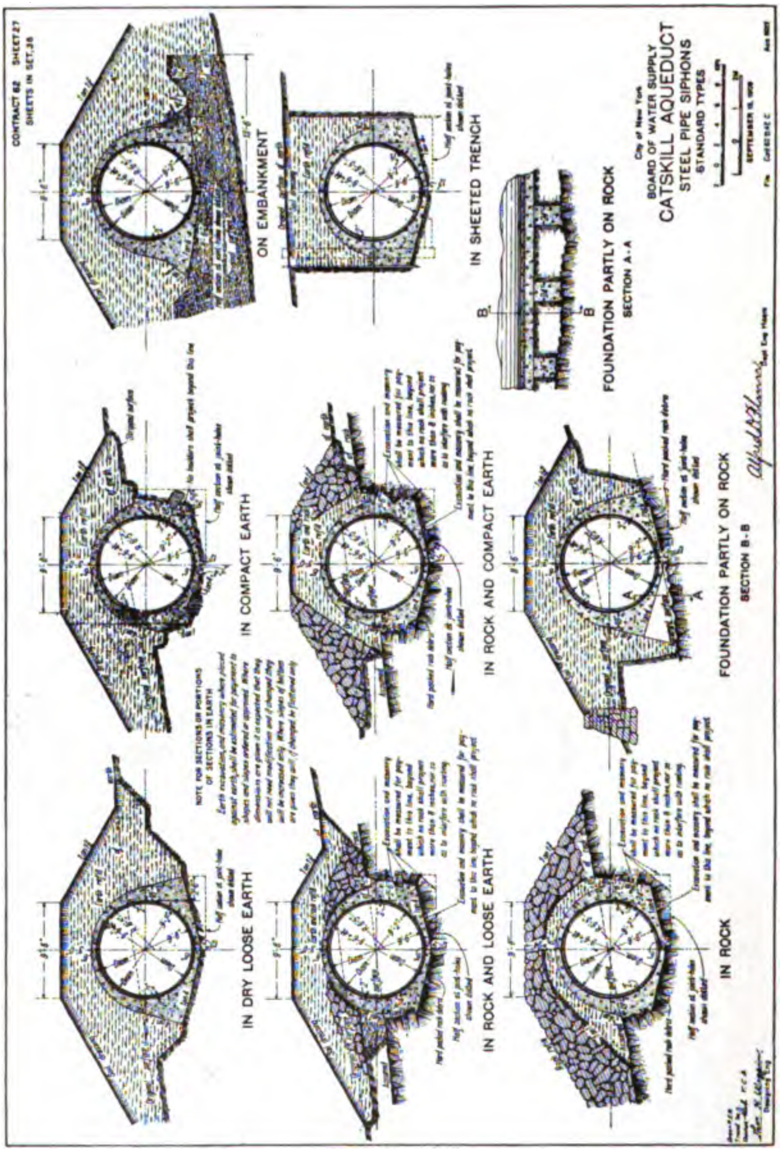

FIG. 1.—STANDARD TYPES OF TRENCH EMBANKMENT AND JACKET, SIMILAR FOR ALL SIZES OF PIPE.

rust and the dirt. Two vertical wooden tanks of suitable diameter were placed side by side, one containing hot dilute sulphuric acid kept at a strength of about 5 per cent of oil of vitriol, which was approximately 93 per cent pure sulphuric acid; after 15 minutes in this solution at a temperature of about 125 deg. F., each plate was of a uniform clear steel gray color all over and when removed was at once dipped into clean hot water in the adjacent tank. Riveting followed pickling quickly, the pipes being made in 15-ft. sections of 2 rings of 7½-ft. net length each. Each ring was one plate, except in the 11-ft. 3-in. pipes, and in those made of plates thicker than ½ in., for which two plates were used. It has been observed that where steel is exposed to corrosion with the mill scale on the surface, the corrosion tends to concentration at certain points and is accompanied with pitting; hence the care exercised to remove the mill scale thoroughly.

Observation and such information as was available indicated that cement mortar adhered most strongly to steel and afforded the best protection when it could be applied directly to the clean surface of the metal. Having obtained a satisfactorily clean surface in the shop, there remained the problem of preserving this surface in as good condition as practicable until the pipes could be laid, covered with the concrete, and lined with the mortar. Various temporary coatings were suggested, but it was finally decided to use whitewash, and so each pipe was given a coat of heavy lime whitewash before it left the shop. To each barrel of whitewash, about 50 gal., there were added about 20 lb. of glue; the glue was dissolved in water before mixing in the whitewash. After this mixture had been used for a time, about 1 lb. of Portland cement was added for each gallon of whitewash. This whitewash, applied with brushes, did not adhere very well, and through lack of care in handling, the pipes suffered more or less almost as soon as it was applied. They suffered more from exposure to the weather, however, than from abrasion; but even where the whitewash was not disturbed light rusting occurred. Only around the rivets and at joints where the whitewash had formed a very thick coating was there no sign of rusting. Hence, as delivered at the trench, the pipes had more or less complete coats of light yellow rust very uniformly distributed on the bared portions of the steel, without indications of any tendency to

pitting. This light rust has been regarded as not seriously objectionable.

Probably the greatest obstacle to securing the desired protection of the steel between the time it leaves the shop and the time it is covered with the mortar or concrete is the tendency of the steel mill and fabricating shop to push their operations much more rapidly than the pipe can be laid, tested, and covered, and the uncertainties of the field work, which make it desirable to have pipe on hand ahead of the progress of laying. These circumstances resulted in some cases in the gradual formation of heavier rust; this has been removed, as required by the specifications, as is also the whitewash which still adheres to the pipes, just before the applications of the jacket or lining.

Ease of removal and probable lack of objectionable effects upon the mortar, if small quantities should not be removed, were among the reasons for adopting whitewash. Unquestionably, Portland cement grout would have stuck more tenaciously, but it would have been correspondingly difficult to remove, and it was thought probable that the mortar of the lining and jacket would not adhere well to the old cement surface; these were considered sufficient arguments for forbidding its use. For removing the rust and dirt from the pipes, wire brushes are commonly used, and in some of the worst places steel scrapers also. Inside some of the siphons the surfaces have been rubbed with empty cement bags after the wire brushing. This final cleaning is done in short stretches just in advance of the placing of the concrete or mortar.

To support the pipe in the trench so as to permit the placing of the concrete jacket beneath it, and also aid in bringing the pipe to line and grade as it was being laid, concrete blocks called cradles were built in the bottom of the trench, Figs. 2, 3 and 4 show the several styles. At first attempts were made to have these cradles fit the bottom of the pipe closely, but this gave trouble and the shape was modified. Some cradles were made about 24 in. square and extended about 6 in. below the ordinary sub-grade of the trench, with their tops at the proper grade for the bottom of the pipe, and, on the whole, this shape was satisfactory. Some of the longer cradles were cracked, due probably to unequal bearing or shocks in placing the pipe. Most of these

cracks were due primarily to the uneven bearing of the pipe on the cradles.

The pipe having been laid, riveted and calked was filled with water to hydraulic gradient, inspected, and leaks further calked. Bulkheads were placed in the open ends of the pipe and a small riser pipe carried up to the proper elevation to represent

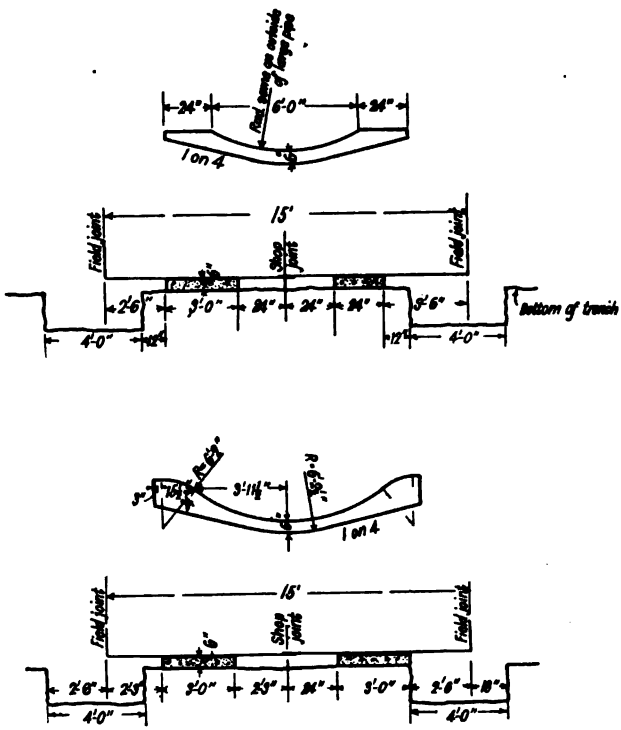

FIG. 2.—CONCRETE CRADLES FOR PIPE SIPHONS.

the working head when the aqueduct would be in service. To maintain this pressure on the pipe constantly, a small reservoir or tank was attached to the top of the riser pipe and kept just spilling over. While the pipe was still full of water under this normal working pressure, the concrete jacket was placed about it by methods similar to those used in building concrete conduit, excepting, of course, that no inside form was needed, Figs. 5 and

6. The water pressure was continued until this concrete had attained considerable strength, the period depending upon the weather, kind of cement, and other conditions. It was found undesirable to maintain the water pressure by direct pumping into the pipe while the concrete was being placed and was hardening, since fluctuations of head caused a few cracks. Maintaining this pressure by the small overflow tank and riser pipe mentioned above was more satisfactory. When the last concrete was suffi-

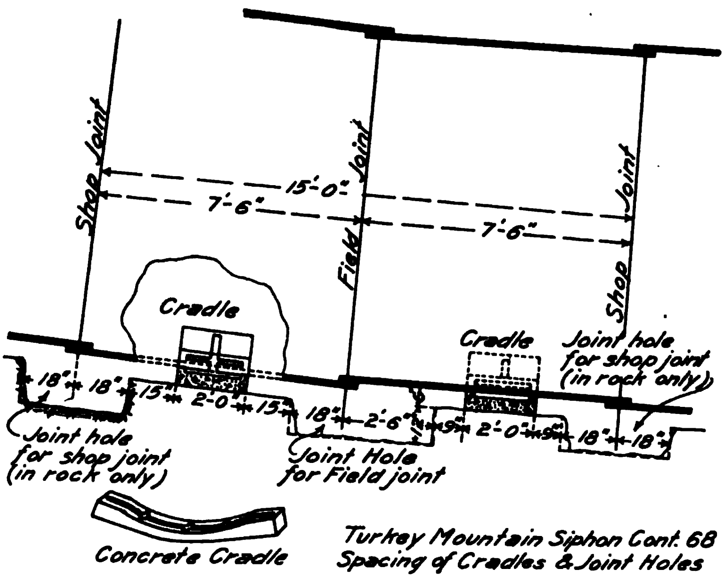

FIG. 3.—TYPICAL ARRANGEMENT OF CONCRETE CRADLES AND JOINT HOLES FOR FIELD RIVETING.

ciently hardened, the water was slowly withdrawn from the pipe.

Three variations in the procedure of placing the concrete jacket were tried: (1) Monolithic, as specified; (2) first the invert, then the remainder, and (3) first invert and side walls to the horizontal diameter, and then the arch. The best contact between the jacket and the pipe seemed to have been obtained when the concrete was placed monolithically. It was found unwise to permit the concrete to be dumped from the buckets with a greater

drop than 2 ft., because the vibration tended to crack the concrete recently placed.

On one siphon the average rate of placing concrete jacket was about 130 ft. a week; on another, 170 ft.; and on a third,

FIG. 4.—LAYING STEEL PIPE ON CONCRETE CRADLES.

190 ft., the maxima on these three siphons being, respectively, 223 ft. per week, 297 ft. and 260 ft. On one siphon the progress of 30 ft. of full section per 8-hour shift, using about 50 yd. of concrete mixed by machine was attained. Many factors influ-

enced progress, such as the method of mixing, method of transportation whether by cableway, wheelbarrows or cars, and the style of outside forms, whether steel or wood, and the conveniences for moving and setting the forms. Earth covering was placed in many cases immediately after the completion of the concrete jacket, but, in a few instances, considerable time elapsed before the concrete was covered. So far as observed, there have been few cracks in the concrete jackets built to the end of last season.

FIG. 5.—METHOD OF PLACING CONCRETE JACKET AROUND STEEL PIPES.

Two quite different methods were at first tried by the two contractors for lining pipes. Under Contract 62, the contractor began at once by grouting with forms; under Contract 68, an attempt was made to use the cement gun, and the Hunter's Brook siphon, 1493 ft. long, 9 ft. 9 in. diameter of pipe shell, was lined in this way. The cement gun* was fully described in *Engineering Record*, July 1, 1911, and other descriptions have been published, so that it need not be described again in this paper. Several

* See *Proceedings*, Vol. VII, p. 504.—ED.

machines are shown in Fig. 7. Charges of dry sand and cement mixed in prescribed proportions were placed in the charging chamber of the machine and then dropped into the pressure chamber. From the latter the mixture was rapidly discharged under an air pressure of 50 to 60 lb. per sq. in. through a rubber hose, while through a parallel hose joining the former in a special nozzle, water under pressure was discharged so that the sprays of water and of sand and cement were commingled. A pressure of about 30 lb. was maintained at the nozzle and so the mixture

FIG. 6.—ONE OF THE CONCRETE MIXERS.

was thrown with considerable force against the surface being coated. This aided in securing an excellent union between the mortar and the steel and more especially between the successive portions or layers of the lining. The operator held the nozzle 2 or 3 ft. away from the surface being coated, moving it back and forth continuously, meanwhile controlling the discharge from the nozzle by lever valves.

The force with which the mixture of sand, cement and water is thrown against the surface being covered causes a measurable

proportion of the sand grains to rebound from the surface and fall into the bottom of the pipe, particularly when beginning the first layer on the bare steel. By analysis this dry material was found to contain about 1 part of cement to 3¾ parts sand; it was collected and used for making the invert or bottom part of the pipe lining, which was deposited as a mortar and screeded to shape in advance of the gun work. Because of this separation of sand, an excess of it is put into the dry mixture in order to prevent the lining being richer than intended. Another result

FIG. 7.—FOUR CEMENT GUNS ON HUNTER'S BROOK SIPHON.

is that the atmosphere inside the pipe is commonly very dusty and in order to make reasonable working conditions for the men artificial ventilation was resorted to. Furthermore, the mortar thus applied contained less water than the grout, and more attention must be given to keeping the lining moist in order to minimize shrinkage cracks. Fine cracks have formed rather numerously and are probably due, in considerable measure, to the failure to keep the lining wet enough while it was setting and hardening. In spite of the difficulties, a lining which up to date seems satisfactory was secured. When this siphon was completed, the use

of the cement gun was discontinued, the contractor giving as a reason excessive cost. Lining by the grouting method was then adopted under Contract 68.

It has been common practice to place the lining in the invert of the pipe by screeding, the width thus placed varying from about 2 to 6 ft. measured along the arc. Practical difficulties prevented the use of a simple complete cylindrical form; among these difficulties were going around curves, either vertical or horizontal, collapsing one set of forms sufficiently to pass it forward through another set which must remain in place while the grout was hardening, and cleaning, lubricating and inspecting the outside of the form inside the pipe. Consequently, forms in panels about 2 ft. wide and 15 ft. long (7½ ft. at curves) were adopted. These are supported on wooden ribs or centers adjusted to give the proper thickness of lining and firmly braced, Fig. 8. The lining has been commonly poured in 15-ft. sections in one operation.

The grout is poured from outside the pipe through a 2½-in. wrought-iron pipe secured into a rivet-passing hole. In the Southern Department this pouring pipe is at the downhill end of the section and is long enough to give a head of about 4 ft. on top of the uphill end. A vent pipe is fastened in the uphill rivet-passing hole of the section, the bulkhead forming the end of the lining being placed just below it. For pouring grout the contractor for the northern siphons has always used two mortar boxes set on a temporary staging over the upper end of the section to be grouted. The mortar is mixed to the proper consistency in alternate boxes and is allowed to flow into the pipe through a hole controlled by a sliding wooden gate. All mixing is done by hand, and materials are carefully graded. It generally takes about two hours to fill a section, after which a man is kept on for an hour or two in order to feed in sufficient grout to get the desired consistency. There is a noticeable tendency to get a porous or thin condition at the upper end of the section, near the grouting hole. To avoid this, the riser is removed two or three times during the pouring and the thin material which collects at the top is allowed to escape. The riser is then put back and grout added until desired results are obtained. In some cases it takes nearly two hours after the main operation to get grout of proper con-

sistency at the foot of the riser. At the finish of the pouring a small pipe is inserted through the large pipe, to permit the escape of the last air while grout is poured through the larger pipe and churned into the small remaining space, to insure complete filling. For the first batches, or nearly up to the horizontal diameter of

FIG. 8.—WOODEN FORMS IN PANELS FOR PLACING CONCRETE LINING BY THE GROUTING METHOD.

the pipe, the grout is mixed 1 part cement to 1 part sand, and the remainder about 1 to 2.

Some fine cracking has occurred in the lining placed by the grouting method as well as in that deposited by the cement gun. None of this cracking is believed to be serious. As an aid to preventing cracks and a safeguard against the remote possibility of small pieces of the lining becoming loose and falling out, both contracts provide for using wire fabric in the mortar, as a rein-

forcement, but up to the end of last year none had been used. Permanent absolute adhesion of the concrete or mortar to the steel is not being attained at all parts; this has been proven by careful sounding with a hammer, but on cutting into hollow sounding spaces, the space between the mortar or concrete and the metal has been found almost infinitesimal in width. Before preparing the contracts, such an occurrence was anticipated and some tests were made at the Board's laboratory to help in determining how serious a matter this would be. A brief statement of these experiments follows:

One experiment was made as follows: Four circular slabs 15¼ in. diameter and 3 in. thick were made of concrete in proportions 1 cement: 2.7 Jerome Park screenings: 6.3 Jerome Park stone (gneiss) by weight, the consistency of the mix being rather dry. Carefully imbedded in the center of the slabs were four ¾-in. round, soft-steel rods spaced 3 in. apart, so that there was at least 1½ in. of concrete in all directions from each rod. When the concrete had set, two of these slabs were immersed in water in two tanks 1 ft. 6 in. in diameter and 2 ft. deep. The other two slabs had two galvanized-iron cylinders 15¼ in. in diameter cemented to them, and a head of 20 in. of water maintained on them. The water in the tanks and cylinders was kept at a constant depth and head by the addition of water whenever it was necessary. The slabs subjected to percolation leaked rapidly at first, but became gradually tighter, and during the last few months of the tests there was very little leakage. The tests were conducted in open air, and were subject to the variations in outdoor temperature. The tests commenced July 13, 1907. On March 23, 1909, one year and eight months later, the slabs were broken and rods examined for corrosion. It was found that the protection of the rods by the concrete was perfect in all cases, there being no sign of corrosion on either the black finish of the metal left by the rolls or on the bright ends of the rods exposed in cutting to length with the hacksaw. The concrete when broken was found to be thoroughly saturated, showing that the water had full access to the rods.

Another experiment was conducted as follows: Six steel plates 8 in. by 16 in., of 12 gauge, cleaned by pickling and then by rubbing with emery cloth, were placed horizontally in a galvanized

iron tank 18 in. in diameter and 24 in. deep, separated from the bottom by two $1\frac{1}{4}$-in. bars of alberene stone, and from each other by $\frac{1}{2}$-in. wood dowels. The first pair of plates was put in without any protective covering. The second pair had their upper surfaces protected by a slab of cement mortar $2\frac{1}{2}$ in. thick, not in contact with the steel, but separated from it by two metal strips about .04 in. thick. The third pair of plates was protected by cement mortar slabs 2 in. thick, cast directly on the steel, and apparently adhering to it firmly. The tank was then filled with Croton water to a depth of 4 in. above the top of the uppermost mortar slab and kept filled the entire duration of the test, the water being renewed twice monthly.

After two years' immersion, the plates were taken out and cleaned off by washing with a sponge. The first pair of plates showed heavy corrosion. In numerous places the entire layer of oxide had separated from the steel and formed blisters, leaving the bright steel surface underneath. The second pair of plates showed a very slight corrosion. Most of this washed off, thus indicating a considerable protective influence of the mortar slabs even when separated from the metal by a space of .04 in., open at the edges all around. When the mortar slabs were removed from the third pair of plates, it was found that a part of the surface of the steel had a distinctly different appearance from the other part. One part was clean and wet; the other part was covered by strongly adhering particles of mortar and was dry, thus indicating that there had been actual adhesion of the mortar only over the latter part of the surface and that the former had been separated by a space large enough for the water to enter. There had, however, been no rusting except at some places near the edges of the wet part of the surface, where apparently the space had been big enough to allow circulation of the water. The water in the rest of the space had evidently been so highly charged with lime that no corrosion could take place.

EXTRACTS FROM CONTRACT 62.

GENERAL SECTIONS.

Order of Work.

SEC. 19. The steel pipe shall be laid, tested, and made tight against hydrostatic pressure; then surrounded by concrete while still under the normal hydrostatic pressure, after which the mortar lining shall be placed. Except at overhead stream crossings, no stretch of pipe shall be left not well protected from frost between stretches of pipe around which concrete has been placed. As soon as practicable after the concrete covering and mortar lining have been placed, the pipe shall be covered with earth; and all pipe covered with concrete, whether or not mortar lining had been placed in it, shall be protected in freezing weather by at least one foot of earth.

MORTAR LINING FOR STEEL PIPES.
(Item 26.)

Work Included.

SEC. 26.1. Under Item 26 the contractor shall build a mortar lining inside the steel pipe and cast-iron bell castings as specified, directed or approved.

Description and Proportions.

SEC. 26.2. The lining shall consist of Portland cement and sand, mixed in ordered proportions, probably one part of cement to two parts of sand. The quality of the sand shall be as specified under Items 28 to 30. Reinforcement, if used, will be paid for under Item 27. The lining shall be of substantially uniform thickness throughout the entire circumference except for the unavoidable variations due to lap of the plates, butt straps, and rivet heads. The thickness over the inner course of plates is to be 2 in., that is, the internal diameter of the lining shall be 4 in. less than the nominal diameter of the steel shell.

Forms.

SEC. 26.3. Forms shall be of steel, or of wood covered with galvanized sheet steel, and shall be especially constructed so as to have sufficient strength and yet be adjustable so as to give a uniform space between them and the shell of the pipe. Great care shall be exercised to secure forms which will leave the surface of the lining perfectly smooth. Forms which give unsatisfactory results after use shall be satisfactorily repaired or replaced. The length of the sections of forms will not be restricted provided satisfactory means are adopted for controlling the uniform thickness of the lining and the correct spacing of the reinforcement, but sufficient sections shall be provided adapted for lining the pipe on curves, where lining shall be placed in sections about 7 ft. long. Each time the forms are used they shall be thoroughly cleaned and then coated with some approved inadhesive substance which will prevent the mortar from sticking to the forms without injuring the mortar. The lining of manhole

castings, and of the blow-off elbows to the sockets, shall be monolithic with the lining of the steel shell, and forms shall be so constructed as to permit this method.

Method of Placing.

SEC. 26.4. The lining shall in general be placed by pouring a grout around an internal form, through holes cut in the top of the pipe for that purpose in the manner and at locations specified in Section 19.11. Sections shall be so terminated as to bring a hole at the upper end which shall be arranged as an air vent. The mortar shall be mixed to a thick creamy consistency and allowed to flow into place as uniformly as possible. When the section is filled to the top, the pouring of the grout shall be continued until grout runs from the air vent; then headers of steel pipe shall be screwed into the inlet and outlet holes and filled with grout so as to put a head of at least 4 ft. on the highest part of the section, and these headers shall be kept filled with grout until the grout has set, when the pipe shall be removed and the hole made watertight by a screw plug. During the pouring of the grout, the form shall be tapped to loosen air bubbles, and a careful watch shall be maintained to prevent leaks. The work shall be so planned that the grout can be poured continuously from start to finish of the section. Any interruption greater than fifteen minutes, whether due to leaks or any other cause, may be sufficient reason for the rejection of the entire section.

Lining with Plaster.

SEC. 26.5. Should there be any portion of the interior of the pipe which it is impracticable to line by grouting, this portion shall be lined by plastering the pipe with mortar, mixed as specified in Section 26.2. Only skilled masons or plasterers shall be allowed to do this plastering, and a section once started shall be prosecuted until finished, with only such pauses as are necessary for a sufficient setting of a layer to permit the next layer to be placed. Each layer shall be as thick as is feasible to apply, so that as few layers as possible may be necessary. The surface of each layer, except the final one, shall be brushed to thoroughly remove the laitance and then deeply scratched or otherwise satisfactorily treated to give a bond with the succeeding layer.

Removal of Forms.

SEC. 26.6. The forms shall be removed within twelve hours of the time set by the engineer, and the section, if accepted, shall receive immediately such repairs as required, in the manner directed. It is possible that the lower part of the lining will, in many cases, show a sandy surface, and if so, it shall be brushed with enough neat cement wash to fill the pores and no more, and troweled to a smooth finish. Any section not accepted shall be immediately removed by the contractor at his own expense and replaced by acceptable lining.

Prevention of Freezing; Bulkheads.

SEC. 26.7. Suitable bulkheads shall be erected in the pipe to prevent freezing inside the pipe either during the placing of the lining or after its

completion. They shall be removed before the completion of the contract, if ordered. Lining shall not be placed in the uncovered pipes over streams during, or within a month before, freezing weather, unless the pipe is satisfactorily protected, and, after lining these portions of the pipe, water shall not be allowed to stand and freeze there.

Measurement and Payment.

SEC. 26.8. For placing the lining in the steel pipe and cast-iron bell castings, the contractor shall receive the price per linear foot of pipe stipulated, the measurement to be made along the axis of the pipe, this price to include all labor and materials necessary to complete the lining in a thorough and approved manner except only that the cement required will be paid for under Item 35 (Portland cement).

REINFORCEMENT OF MORTAR LINING FOR STEEL PIPES.

(Item 27.)

Description.

SEC. 27.1. Reinforcement may be ordered under Item 27 for any part or the whole of the mortar lining. The reinforcing material shall be galvanized steel mesh of a style and weight approved, provided, however, that no reinforcement shall be required of which the lowest price obtainable by the contractor, f.o.b. New York City, exceeds $\frac{3}{4}$ cent per square foot, for lots of 10,000 sq. ft.

Placing.

SEC. 27.2. The reinforcement shall be placed approximately in the center of the mortar lining. The reinforcement may be kept away from the pipe by distorting the reinforcement at frequent intervals, so as to make points projecting toward the pipe. Unless otherwise permitted, small blocks of mortar shall be attached to the reinforcement, for the purpose of keeping it away from the form. Metal shall be lapped at least 6 in. at all longitudinal joints.

Measurement and Payment.

SEC. 27.3. The quantity to be paid for under Item 27 shall be the number of square feet of lining, measured as of a mean diameter 9 ft. 4 in., in which reinforcement has been ordered and placed. This does not include any allowance for lap. The price stipulated shall include the cost of the reinforcing metal, royalty if any, cutting, shaping, bending, wires, clips, mortar, and other devices used for holding the reinforcement in place, or for splicing the strips; and it shall further include any additional expense of forms, tools, appliances, and labor other than the expense that would be required for finishing the mortar lining under Item 26 without reinforcement.

The gain in smoothness of interior by covering the rivet heads and the plate laps has been computed to so increase the hydraulic capacity that three pipes equal four without lining. The total cost for the siphons with three lined and jacketed pipes is estimated at about the same as for four pipes constructed and coated in the more usual way. Obvious incidental advantages are secured by the more permanent construction.

The Board of Water Supply of the City of New York is

TABLE I.—CONTRACT PRICES FOR STEEL PIPES, CONCRETE JACKET AND MORTAR LINING.

Description.	Contract Price.	Average of All Bids.
Contract 62.		*(5 bidders.)*
9 ft. 6 in. steel pipe, 7/16-in. plate, lap jointed	$31.00 lin. ft.	$35.50 lin. ft.
9 ft. 6 in. steel pipe, 1/2-in. plate, lap jointed	35.00 " "	41.20 " "
9 ft. 6 in. steel pipe, 1/2-in. plate, longitudinal seams butt-jointed	40.00 " "	44.80 " "
9 ft. 6 in. steel pipe, 9/16-in. plate, longitudinal seams butt-jointed	43.00 " "	48.80 " "
9 ft. 6 in. steel pipe, 11/16-in. plate, longitudinal seams butt-jointed	47.00 " "	55.80 " "
9 ft. 6 in. steel pipe, 3/4-in. plate, longitudinal seams butt-jointed	50.00 " "	50.60 " "
Mortar lining for steel pipes	2.50 " "	4.90 " "
Reinforcement of mortar lining for steel pipes	.02 sq. ft.	.03 sq. ft.
Concrete masonry around steel pipes	6.00 cu. yd.	5.85 cu. yd.
Portland cement	1.75 bbl.	1.74 bbl.
Contract 68.		*(8 bidders.)*
9 ft. 9 in. steel pipe, 7/16-in. plate, lap jointed	$29.00 lin. ft.	$33.25 lin. ft.
11 ft. 3 in. steel pipe, 7/16-in. plate, lap jointed	33.00 " "	37.37½ " "
11 ft. 3 in. steel pipe, 1/2-in. plate, lap jointed	38.00 " "	42.75 " "
11 ft. 3 in. steel pipe, 1/2-in. plate, longitudinal seams butt-jointed	46.00 " "	48.75 " "
11 ft. 3 in. steel pipe, 9/16-in. plate, longitudinal seams butt-jointed	50.00 " "	53.62½ " "
Mortar lining for 9 ft. 9 in. steel pipe	3.00 " "	3.56¼ " "
Mortar lining for 11 ft. 3 in. steel pipe	3.50 " "	4.04½ " "
Reinforcement of mortar lining for steel pipe	.02 sq. ft.	.02¾ sq. ft.
Concrete masonry around steel pipe	5.25 cu. yd.	5.78 cu. yd.
Portland cement	1.60 bbl.	1.72½ bbl.

constructing the Catskill Water Works. Engineering operations are being directed by J. Waldo Smith, chief engineer; Robert Ridgway was department engineer, Northern Aqueduct department, until January 14, 1912; Ralph N. Wheeler was appointed department engineer of that department, February 1, 1912; Frank E. Winsor is department engineer, Southern Aqueduct department. In immediate charge of the construction of the siphons are division engineers John P. Hogan, Alexander Thomson, Jr., and George P. Wood (Northern); George G. Honness,

Ernest W. Clarke, and Charles E. Wells (Southern). The drawings and specifications were prepared and many of the preliminary investigations conducted by Senior Designing Engineer Thomas H. Wiggin, and Engineer Inspector Ernst F. Jonson has had charge of inspection of cement, of steel at the rolling mills and of pipe at the shops, all under the immediate supervision of the writer. The chief engineer and his personal assistant, Department Engineer Thaddeus Merriman, made many examinations of existing steel pipes which furnished the reasons for seeking a better protection than the usual coatings.

All drawings reproduced as illustrations are for the 9-ft. 6-in. pipes. Corresponding drawings for the other sizes are similar; likewise standard dimensions for rivet and joint details.

A FIREPROOF SCHOOL OF CONCRETE.

By Theodore H. Skinner.*

Early in the winter of 1910–11 the writer was intrusted by the Trustees of Union School District No. 12, Town of Vernon and City of Oneida, N. Y., with the task of building a new four-classroom school house which should, in addition to complying with all the regulations of the New York State Department of Education, be as nearly fireproof as possible, keep within an appropriation of $17,000, be so arranged as to appear symmetrical and complete, while in reality be only one-half of an eight-room building ultimately desired. The school house shown by the accompanying illustration is the answer to the many problems involved in carrying out the task.

The general plans were gone over with the state inspectors for the purpose of securing informal approval of same before detailed drawings and specifications were completed. These plans were then submitted to several parties with requests for estimates and sketches showing how they would build the framework of floors, roof and enclosing walls. These parties represented field-cast reinforced concrete, light steel frame with metal lumber and metal lath stuccoed and factory-made reinforced concrete.

The sketches received were carefully studied and general drawings made which would be possible to follow should either system be selected. Tenders were then invited from a number of general contractors. The specifications provided that the bidders might use either one of the three systems proposed and asked them to name in their bid what would be the difference in cost of the building if erected by them under the various systems. The bids ranged from $16,298.00 to $24,500.00 for the bare building without plumbing or heating and nothing to spare for moving furniture, etc. The local contractor, who proved to be lowest bidder, then opened his estimate books for inspection and it was found that by combining his own figures for portions of the work

* Architect, Oneida, N. Y.

with the figures named by the party bidding on factory made reinforced concrete that it would be possible to carry out the general plans and keep within the appropriation. Accordingly detailed specifications and drawings were made up for the framework of the building and the general contract let.

The building (Figs. 1 and 2) consists of a rectangular section 25 x 92 ft. running north and south, containing four classrooms each 24 x 32 ft., four coatrooms 7 x 24 ft., stairway 11 ft. wide, two playrooms in the basement 24 x 32 ft., separate toilet rooms

FIG. 1.—KENWOOD SCHOOL, KENWOOD, N. Y.

each 7 x 24 ft. for boys and girls and a teachers' room 11 x 13 ft. with private toilet on the landing over the first- and second-story stairway. An extension across the front 13 x 40 ft. contains furnace room in the basement, and is entirely corridor on the two upper floors. There are two minor further extensions forming vestibules at front and rear.

The plans (Fig. 3) provide for two additional classrooms at both the north and south ends of the present rectangular section to be reached by extensions of the present corridors across the west or blank sides of the present classrooms. Additional stair-

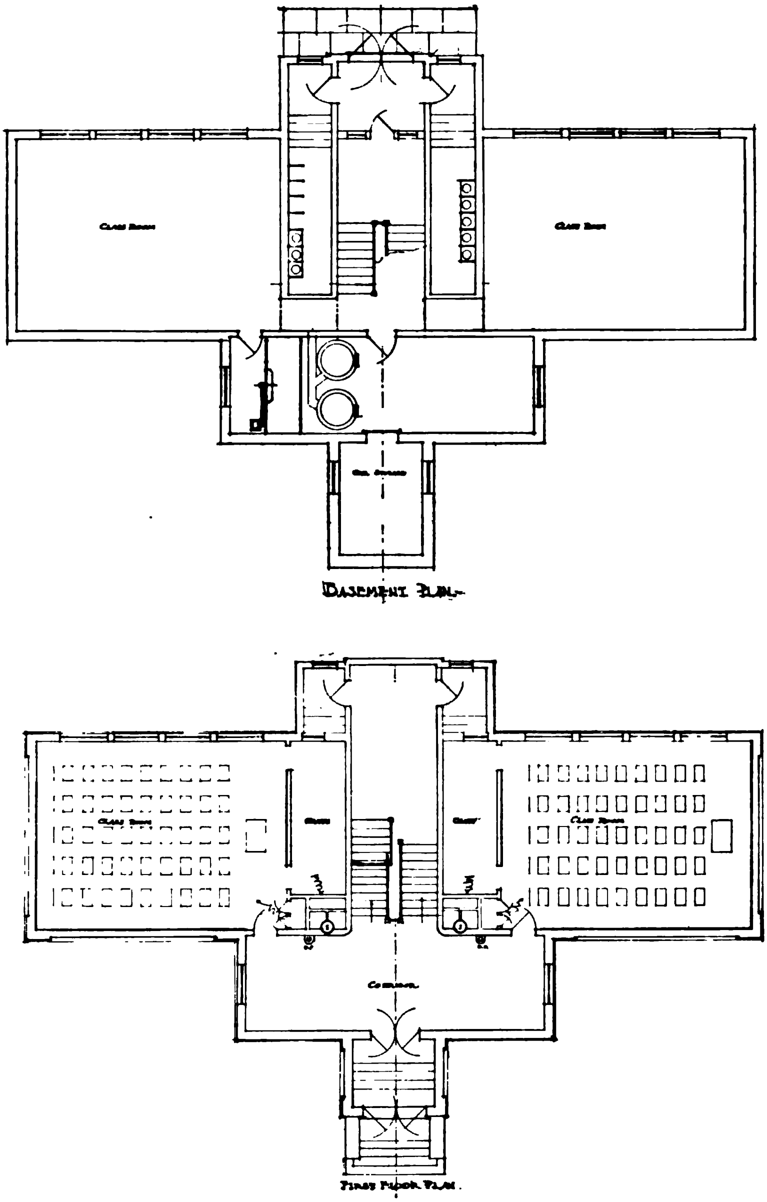

FIG. 2.—GROUND AND FIRST FLOOR PLANS, KENWOOD SCHOOL.

ways will be provided in the extended corridors and also coatrooms for the new classrooms. When these extensions have been made the west or street face of the building which now presents large blank panels will be symmetrical on the center line of the building and when finished will have large windows indicating the lines of the stairways.

The present classrooms have southeastern exposure with unilateral lighting only. The coatrooms are accessible only from the classrooms. Each classroom is provided with a built-in cabinet of four drawers and two cupboards for the teachers' use and has blackboards 4 ft. high entirely around the walls excepting for the window spaces. The basement rooms are 10 ft. in the clear, the classrooms 12 ft. The plumbing fixtures are all of colonial ware and of the latest pattern, two drinking fountains are arranged in the corridors, one in each story. Special anti-panic exit bolts are arranged on the outside doors, which open out.

The details provided for a steel frame for the classroom section, of 12 columns connected by horizontal steel girders in the plane of the walls at several points. The girders carried 2 floors or 2 sets of separately molded and cast reinforced concrete joists, spaced about 4 ft. on centers which rested on and were anchored to them. The joists carried in turn a series of ribbed reinforced concrete slabs separately molded and cast in the factory (Fig. 4). The roof was constructed in same manner as the floors, the only difference being that the joists were not of uniform section throughout being severally graded or warped so as to give the roof slabs resting on them the proper pitch to throw the water to desired points. The floors and roofs of corridors and entrance porches were built also of separately molded members supported by bearing walls of masonry. The type of unit members employed is shown in Fig. 5.

A short description of the process of manufacturing separately molded reinforced concrete members may be of interest. A frame was made up of the steel rods necessary to reinforce the concrete, carefully designed to take care of all the tensile stresses which might be developed in the member when finished, set in place and loaded, and also all shear which the concrete would not take care of. Longitudinal tension rods were carefully bent to

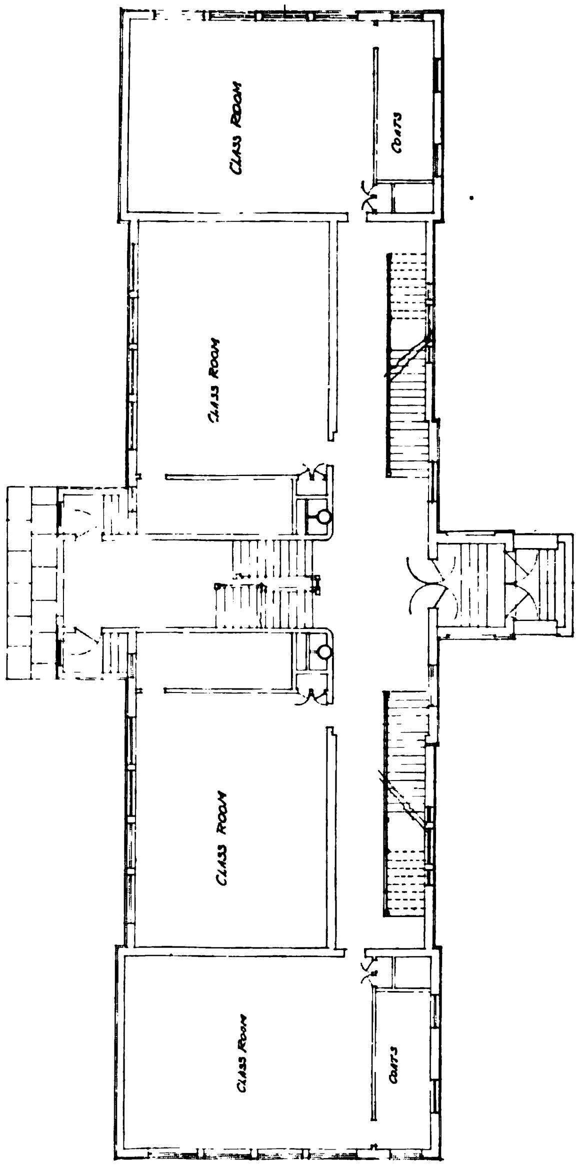

FIG. 3.—PLAN SHOWING FUTURE EXTENSION, KENWOOD SCHOOL.

the desired form and united into rigid frames by means of vertical loops and small rods wound around the larger ones which also take care of the shear. Some smaller longitudinal rods were built into the frames at their tops to take care of any excessive compression or any tension which might be caused by negative bending moments produced in handling the beams. Special loops were attached to the tension rods at their quarter-points extending up above the top rods in the frames and above the finished concrete at the top by which the beams were lifted, the strain all coming on the reinforcement.

FIG. 4.—REINFORCED CONCRETE SLABS IN YARD OF CONTRACTOR.

Sand molds were prepared on large casting floors of the shape desired for the finished beams and in these molds the unit reinforcing frames were suspended in proper place, and then liquid concrete was poured into the molds and thoroughly worked in and around the reinforcement. The molds were filled to the top and struck off with the straight edge. A mixture of 1 : 2 : 4 concrete was used, the largest aggregate passing a 1-in. screen.

The beams were allowed to remain in the sand 7 days after which they were lifted by the loops, carried by a traveling crane into an open yard and stored until ready for shipment.

The manufacture of the floor panels would not have been possible without a vibrating machine. A frame of small rods to act as reinforcing for the webs was first made, this was then covered with wire reinforcement well wired to the rods. The frame was then placed in molds on vibrating machines, the concrete poured and vibrated.

The beams were about 10 in. wide, 18 to 24 in. deep, of Tee section with the top edge rebated on each side to receive the floor slabs. All joists were approximately 25 ft. long. Floor slabs were about 4 ft. square webbed or thickened around the edges and once across the middle to 3 in. thick, the centers were thin panels only 1½ in. thick. Joists and slabs were designed to

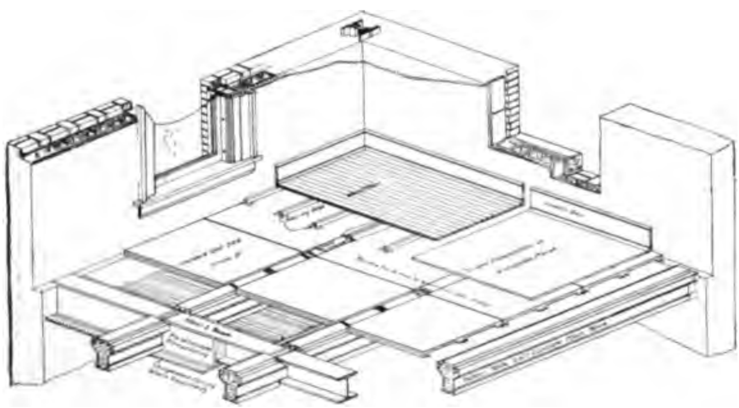

FIG. 5.—GENERAL TYPE OF UNIT CONCRETE MEMBERS.

carry a live load of 100 lb. per sq. ft. in addition to their own weight, with a factor of safety of 4.

The footings for the 12 columns were isolated, 7 ft. square and 16 in. thick. The basement walls between the columns were made of 12-in. hollow terra cotta tile carried by independent footings. The tiles were laid up in cement mortar, plastered with cement mortar on the outside and coated with asphalt up to grade. The exterior walls between columns above the basement, being curtain walls only, were laid up of 8-in. hollow terra cotta tile and were stuccoed on the outside and plastered on the inside. The columns and girders were covered with metal lath and concreted in solidly.

The cornices were supported by a cast stone bed molding, ornamented with egg dart topped out with a 12-in. tile covered with stucco. Some small brick inserts were made just below the cornice to relieve the absolute monotony of the color of the stucco and shallow lines were drawn in the stucco facia and elsewhere while it was still soft to form panels and accents. With these exceptions the exterior is a plain gray floated-finish cement stucco. It will be given a brush coat of some waterproof cement stain or finish before another winter to fill up minor checks and prevent water and frost damage.

All exterior windows were combined frame and sash of metal, glazed with clear double-thick glass in the lower panels and with rippled glass in the upper. Each sash was provided with 2 ventilating sections.

The roof is a flat concrete slab pitching only enough for drainage from the level verge to two outlets near the center line. This was covered with 5-ply slag and composition roofing and is not visible from the ground.

The interior finish was the simplest possible. The masonry walls were plastered with two coats of mortar and left under the float, then painted 4 ft. high with oil paint two coats, and above with water color two coats. No wood trim was used around either doors or windows, but plank jambs of wood were used in the doorways to which the doors were hung by pivot hinges at top and bottom. No wood door sills were used, in short the only wood in the entire building being the door jambs and doors, the molding at the top and bottom of blackboards, cleats for coat hooks in the wardrobes and panels of wood flooring in the center of each classroom.

These panels, 4 in number, were laid of one thickness maple floor, $\frac{7}{8}$ x $2\frac{1}{4}$ in. face over 2 x 4 in. hemlock sleepers, on top of the concrete construction previously described, were oiled on the under side before being laid and given a coat of oil immediately after laying to prevent their absorbing water from the composition borders, subsequently laid against them. The balance of the classroom floors were finished on top of the cement slabs with concrete surfaced with colored material to match the wood center. These borders merged into a sanitary or cove base 4 in. high everywhere. Corridors, stairways and wardrobes all have a

granolithic cement coat over the structural concrete slabs, this finish also merging into a sanitary base 4 in. high. There was no projection to these sanitary bases at the top, the same being finished flush with the finish plaster.

The four classroom ceilings were plastered upon expanded metal lath carried by angle-iron furrings which in turn were

FIG. 6.—CEILING OF PLAYROOM, KENWOOD SCHOOL.

attached to numerous iron lugs left projecting from the cast concrete floor joists for this purpose. These furred ceilings were used for a double purpose, first to make a ceiling free from all shadows, and, second, to provide a deadening air space so that the noise of walking on the top of the concrete construction should not be heard in the room below. The basement, corridor and

stairway ceilings were left unfurred, with the cast joists and panel slab construction showing, as may be seen in Fig. 6, all painted with water color paint presenting a very attractive appearance.

The contract, let May 15, 1911, provided for the completion of all work by September 15, but delays were experienced from the start, first in getting the necessary steel fabricated and later in securing competent labor for erecting the steel and laying the heavy concrete floor and roof joists. It was not until November 26, or a few days over six months from the start, that the work was completed.

The joists and slabs were loaded on cars at the factory and shipped by local freight 28 miles; were unloaded by means of a small traveling crane on to farm wagons and then hauled a little better than a mile to the job. Here they were unloaded and swung into place by seemingly inadequate apparatus, all at less expense than the contractor figured he could erect forms and cast the same number of members *in situ*. The joists and slabs stood shipping remarkably well and reached the job in good condition in spite of the rough handling given by the local train, on which the cars are shunted back and forth at every station a great number of times. Only one joist and four slabs required replacing as they were too green when loaded, 30 days' seasoning should be allowed before shipping.

The straightforwardness of the various operations at the building appealed both to the general contractor and to the architect as it offered opportunity to inspect each member before erection and the job was not littered up nor complicated with the forest of supports necessary for field concrete construction. No unexpected difficulties were encountered and no members failed to fit their respective locations accurately, and everyone connected with the work would be satisfied to repeat the operation again, with the exception possibly of the maker of the cast work, who might wish to add slightly to the original allowance for handling the cast members at the building.

The estimates of cost for the various types of construction not used were perhaps confidential and will not be given in detail; it is sufficient to say that they were higher than those obtained for the construction adopted.

The contracts for the building as erected were as follows:

Steel frame	$1,500.00
Structural concrete floors, roof and stairs	3,000.00
Suspended ceilings, unplastered	225.00
Granolithic in halls and coatrooms	196.00
Colored surfacing and sanitary base in classroom	379.00
Iron stair rails	200.00
General contract for balance of work and materials, including excavation, grading, carpentry, painting, etc	9,545.00
	$15,045.00
Plumbing	767.00
Heating	825.00
Hardware	225.00
Total	$16,862.00

In addition to the above a flowing spring was encountered in the excavation which gave standing water over part of the basement and made it desirable to underdrain same and waterproof basement walls; it also necessitated a change in the plans of the cold-air boxes from beneath the basement floor to overhead. These two items cost respectively $200.00 and $227.97. Some additional fireproofing of steel to that provided under the contract was thought desirable and added at a further expense of $400.00. Cement platforms or walks outside at the entrance cost $38.00, making a total of extras on the work over and above the first contracts of $863.97 and bringing the total cost of the building as delivered by the contractor to the Trustees, $17,527.97.

Not counting the area of vestibules, this building has an area of 2820 sq. ft. on the ground, is 40 ft. from bottom of basement floor to top of roof and contains 112,800 cu. ft. Its cost per cu. ft. is 15.5 cts., per classroom $4381.99 and per pupil as $97.37.

THE PRESENT STATUS OF UNIT CONCRETE CONSTRUCTION.

By James L. Darnell.*

To put it briefly it might be said that the present status of Unit Construction is one of progression. Every day new things are developing about this very logical and rational method of constructing and erecting concrete structures. Within the past two years there has been more progress in both the design and in the field work, that is, the construction and erection of separately molded structures, than in all the time previous.

In heavier structures, such as bridges and viaducts, the engineering forces of the Chicago, Burlington and Quincy Railway Company under the direction of Mr. C. H. Cartlidge, Bridge Engineer, and of Mr. George E. Tebbetts, now bridge engineer for the Kansas City Terminal Railway Company, have done much valuable work and they have pioneered the way for others less advancced. The Chicago, Milwaukee and St. Paul Railway Company under Mr. C. F. Loweth, Chief Engineer, has also done some work in this line, but not to the same extent. Mr. R. E. Gaut, while bridge engineer of the Illinois Central Railway Company, also used this system in the design of highway structures in connection with track elevation work in Chicago. All of this work was successfully done and showed pronounced advantages over ordinary methods.

If this is the "Concrete Age" as most of us fondly believe, it will certainly come to pass that the railroads particularly will have to adopt unit construction methods for their concrete structures because it lends itself with peculiar fitness to railway work of all kinds and shows such marked economies in both time and money, that ordinary or monolithic construction is out of the question. This is shown in the present track work of the Kansas City Terminal Railway Company, where Mr. Tebbetts has adopted this form of construction for every structure along the

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line except those where surrounding conditions are such that the use of some other type was imperative, as the McGee Street Viaduct, where the 124-ft. span required a concrete encased steel girder.

In building construction the progress has been more marked perhaps than in bridges and viaducts. The first recorded example of a building of any magnitude built of separately molded units is that of the two kiln houses for the Edison Portland Cement Company at New Village, N. J., early in 1907.* These two buildings were simple one story sheds of the plainest possible type. No attempt was made to elaborate or to refine the design, as nothing more was necessary than a simple assemblance of columns, roof beams and roof slabs. In these two sheds however, the soundness of the principle of separately molded units was demonstrated.

At about this same time the idea of building construction with separately molded units seems to have been taken up independently in other sections of the country. In the east Mr. E. L. Ransome was developing a system which he is employing successfully up to the present time. Perhaps the most notable example of the Ransome type of Unit Construction is the four story building 60 ft. x 200 ft. built for the United Shoe Machinery Company at Beverly, Mass. This building has successfully met all its requirements and is in every respect equal to and in some respects superior to similar buildings of ordinary monolithic construction.

Mr. Charles D. Watson of Syracuse, N. Y., has also done considerable work along this line, having successfully constructed several separately molded buildings.†

In St. Louis, at about this same time, Mr. Albert J. Meier in conjunction with Mr. John E. Conzelman, at that time engineer for the Corrugated Bar Company, and Mr. C. D. Morely a contractor, were working to perfect a Unit system independently and without knowledge of the work that was being done by others. From the beginning there were two very apparent difficulties to be met and surmounted. The first one was the difficulty in devising or designing connections between the separately molded units

* See *Proceedings*, Vol. IV, 1908, p. 48.—Ed.

† See *Proceedings*, Vol. IV, 1908, p. 97; Vol. VI, 1909, p. 391.

which would be as strong as the units themselves. Second, there were well taken objections on the part of many people to separately molded buildings of concrete on account of the pure and monotonous ugliness of such buildings.

The first difficulty, that of the connections, has been most successfully overcome by the use of interlocking and overlapping reinforcement, broad bearings and improved methods of grouting the connections. The second or æsthetic objection is being gradually overcome both by improved and advanced design and by a process of evolution, in which the engineer in designing is

FIG. 1.—INTERIOR OF WAREHOUSE, NATIONAL LEAD COMPANY, KANSAS CITY, MO.

slowly but surely drawing away from the architectural faults of its monolithic predecessor. The progress is best shown in the illustrations which follow.

The first building constructed with separately molded units by the Unit Construction Company of St. Louis, Mo., was a warehouse for the National Lead Company, erected on West Thirteenth Street, in Kansas City, Fig. 1. This building was built under great difficulties in a very restricted space. It was attempted to mold the units within the building lines, which proved to be very expensive by reason of the neces-

sary rehandling of the units from time to time. Because of the unsightly appearance which would have been presented by the use of concrete units in the front of the building, the National Lead Company at that time wanted a brick face. The rear of the building, however, was built of concrete units and its appearance in a measure justifies the objections of the owners on the score of unsightliness. The different points between the units are very clearly indicated and while the wall on the whole does not present a very beautiful appearance, it is after the lapse of some four years, perfectly serviceable, thoroughly substantial and water-tight. We are certainly proud of this, our first build-

FIG. 2.—CASTING YARD FOR CONCRETE UNITS.

ing, because it is still standing up, apparently more substantial than when built and because it has been so thoroughly satisfactory to the owners.

This first building was so successful that the company was enabled, in competition with various other contractors, to secure a contract for a very much more extensive construction for the National Lead Company in St. Louis. Fig. 2 is a general view of the casting yard laid out for this construction. Fortunately there was in this case plenty of room to spread out in, consequently it was not necessary to cast the units on the ground within the building lines. An extensive plot of ground was avail-

FIG. 3.—SETTING A WALL SLAB.

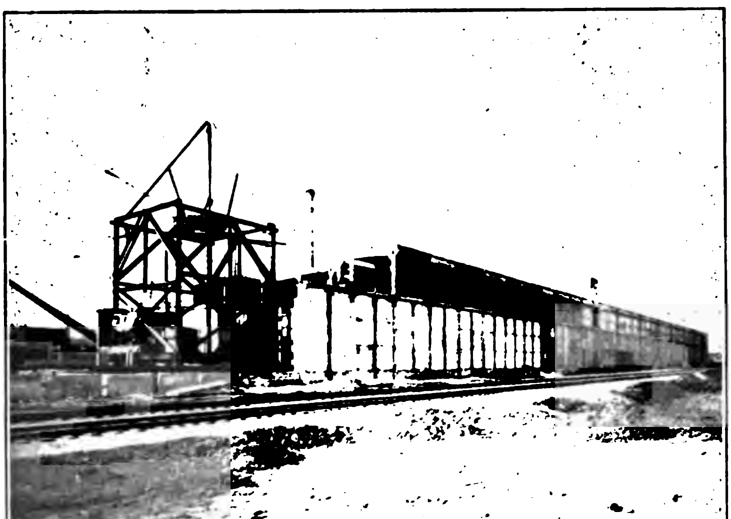

FIG. 4.—ERECTION OF CORRODING STACK HOUSE, ST. LOUIS, MO.

able adjacent to the site of the building and thereon was erected the construction plant. Concrete materials were elevated in a belt and bucket conveyer from cars to bins in the top of the tower, seen in the left center of the picture. Immediately below these bins was the concrete mixer, which discharged by gravity into the cars on the oval elevated track. Movable spouts conducted the concrete from this track level to the molds for the units which were laid horizontally in the casting yard. A traveling derrick was erected to run between the elevated tracks and was used to handle the completed units from the yard onto flat cars for conveyance to the building site.

FIG. 5.—ROOF SHOWING CONCRETE SKYLIGHTS AND LOUVRES.

Fig. 3 is a view of the building after construction had somewhat further progressed. This shows many of the columns in place and the workmen setting a partition slab. It may be seen that the foundations were run in monolithically with slots left for the reception of the columns. A key was cast in each slot to engage a corresponding slot cast in the columns, both of which may be plainly seen in the picture. Fig. 4 shows a view of the structûre when about two-thirds complete. This view illustrates another advantage of unit construction in that as soon as any part of the building is in place it is ready for service. This was peculiarly demonstrated in this particular building. To make room for this structure it was necessary to dismantle and tear

down the old wooden stack building and necessarily the machine capacity was seriously impaired. It proved to be of great advantage that after four or five bays were finished, they were turned over for service. As it happened the company was making white lead in more than three-fourths of the building by the time it was finally completed. Fig. 5 is a view of the roof of this same building, showing the sky lights and louvres, and illustrates the roof construction which was of the regulation felt, tar and gravel type. This building when complete was 750 ft. long, 105 ft. wide and about 45 ft. high.

Fig. 6 is a group of three buildings built for the National Lead Company at their St. Louis plant, contracts for which were

FIG. 6.—BUILDINGS OF NATIONAL LEAD COMPANY, ST. LOUIS, MO.

secured after the stack house had progressed pretty well along toward completion. This group consists of an office and welfare building on the right, an oil house in the center, and a stable and garage on the left of the picture. In this group is seen something of the architectural progress which was referred to earlier in this paper. They certainly present a not unpleasant appearance and we think they appear favorably with some buildings of similar type in any material.

Figs. 7 and 8 represent the most pretentious structure which has engaged the attention of the Unit Construction Company up to the present time. This is an oxide mill also for the National Lead Company, built in St. Louis, which provided for floor loads running from 500 lb. on the first floor to 250 lb. on the top floor.

FIG. 7.—OXIDE MILL IN COURSE OF ERECTION, FIVE STORIES.

FIG. 8.—OXIDE MILL BUILDING COMPLETED.

A study of these views will give to the observer an excellent idea of the construction methods employed as nearly all of the details may be observed.

Fig. 9 shows a corroding stack building erected for the National Lead Company at New Kensington, Pa. They are interesting in that they show architectural progress as well as improvement, both in design and workmanship.

Last year the first contract was taken for a 50,000 bushel

FIG. 9.—CORRODING STACK BUILDING, NATIONAL LEAD COMPANY, NEW KENSINGTON, PA.

grain elevator, Fig. 10, for the Highland Milling Company at Highland, Ill. In this case each slab unit had a column cast on the end of it and the whole was held together by big rods running entirely through the building with large cast washers on the outside. This elevator had nine bins in it; eight were storage bins and the ninth carried the elevating machinery. This structure has been in service for more than a year and the owners of the Highland Milling Company are very enthusiastic advocates of Unit concrete construction because during a season in which

it was very hard owing to weather conditions to successfully store grain, in this elevator not a bushel was lost.

A building under construction for the Ohio Cultivator Company at Bellevue, Ohio is shown in Fig. 11. This building is 200 ft. square, three stories high with a high basement.

FIG. 10.—GRAIN ELEVATOR, HIGHLAND MILLING COMPANY, HIGHLAND, ILL.

As to the use of Unit construction in railway work, Fig. 12 shows a reinforced concrete crossing at Sangoman Street in Chicago, Ill. The work of the Kansas City Terminal Railway Company where concrete units are being used for all the viaducts and subways required in the Terminal Company's improve-

ments, which are now going on, includes some forty structures in all. Fig. 13 shows a unit bridge; Fig. 14 column molds as well as some of the finished columns; Fig. 15 shows an outer deck slab which carries the coping and panel fascia.

The pictures show for themselves the present status of Unit Construction both as applied to buildings and to bridges and kindred structures.

A word as to the "why" of unit construction. At first

FIG. 11.—BUILDING FOR OHIO CULTIVATOR COMPANY, BELLEVUE, OHIO.

glance, to an engineer, the system looks novel and interesting, but of narrow application and of little practical value or utility. Invariably, however, after thorough study the engineer becomes more and more convinced of the practicability of the system and of its broad and diverse possibilities. In fact those who are connected with its application cannot find the time to go into the many virgin fields of construction effort, but up to the present have been compelled to devote all their time and energies to the development of the single field of mill and warehouse con-

struction with short excursions into the grain elevator and railway fields.

For unit construction are claimed all the advantages of

FIG. 12.—CONCRETE BRIDGE CROSSING, SANGOMAN STREET, CHICAGO, ILL.

FIG. 13.—RAILROAD BRIDGE OF UNIT CONCRETE COLUMNS, BEAMS AND SLABS.

permanency, strength and ultimate economy to which structures of reinforced concrete are justly entitled, without any of the disadvantages and uncertainties which in the structure built by

ordinary methods are so often attended by unfortunate results—accidents which are always serious, sometimes fatal.

By this method the designer is sure that no expansion joints

FIG. 14.—UNIT CONCRETE COLUMNS AND MOLDS.

FIG. 15.—UNIT CONCRETE DECK SLAB WITH COPING.

will open up in unexpected places because of improper deposition of concrete or other carelessness of the workmen. Each unit member of the structure is built on the ground under practically

factory conditions. Effectual inspection is assured in all respects. The reinforcement is placed where the engineer designed it to be; each unit is a true monolith without seam or flaw; each unit is properly cured and is prepared in the building; the possibility of getting an imperfect column, beam, girder or slab into a structure is entirely eliminated. In short, the manufacturing conditions of the units make for a maximum of efficiency in all departments with the chance for uncertainties reduced to practically a negligible quantity. In erection the same condition prevails as the mechanical handling of the units is exactly the same as in the handling of steel and with properly designed units it is practically impossible to go wrong on the connections.

Unit Construction today is just beginning to come into its own. Its application is wide and the genius of the American engineer will soon lead to the general adoption of a system which combines in itself the maximum of efficiency, economy and permanency and minimizes the uncertainties contingent on the vagaries of the American workingman—who is principally a foreigner.

DISCUSSION.

Mr. Rudolph P. Miller.—Mr. Chairman, I would like to inquire if the joints must not be filled up on the work and whether any attempt is made to carry the reinforcement from one member to an adjoining member so as to effect continuity? Mr. Miller.

Mr. James L. Darnell.—The joints and connections are all grouted in. The reinforcement overlaps and interlocks, i.e. goes through the connection which results in practically a monolith. In fact, extensive laboratory tests show that the connections are stronger than the units themselves. It has been demonstrated in buildings actually constructed and in service that the connections are the strongest part of the structure. Mr. Darnell.

Mr. John E. Conzelman.—The question just asked would indicate that there was some doubt in the speaker's mind as to the stability or rigidity of the buildings and I want to say that the buildings we have constructed seem to be just as rigid and stable as similar buildings constructed in the ordinary manner. In fact after the joints are poured a unit constructed building has the advantages of continuous action and rigidity characteristic of buildings made by the ordinary or monolithic method. It may be interesting to know that the thin walls used on these buildings have proven satisfactory. We have constructed seven or eight buildings with three and four inch walls which have passed through two winters with no complaints from the owners. Mr. Conzelman.

Mr. E. J. Moore.—The question of shrinkage cracks in floors seems to be important. The speaker referred to the absence of cracks in the floors. Cracks from temperature stresses will no doubt occur at the joints where the different sections are put together and that would seem to make an added problem of waterproofing the joints. Mr. Moore.

Mr. Conzelman.—Mr. Darnell stated that cracks often occurred in concrete structures built in the usual way and that these cracks did not at all times select their location with due regard for the appearance of the structure or the feeling of the builder; in fact these cracks often appear at unfortunate places. Mr. Conzelman.

Mr. Conzelman. It is unusual to see a long concrete retaining wall or building in which there are not some cracks. These cracks (neglecting settlement or other structural causes) are due to two causes, shrinkage of the concrete as it dries out and temperature changes, acting separately or in combination.

A great many of the cracks are due to shrinkage stresses but in unit construction these stresses are practically elminated. This is due to the fact that each unit has hardened before it is incorporated into the building. The unit itself may have some slight internal stresses due to shrinkage, but these are not cumulative and do not affect the complete structure. Each unit takes care of itself.

Temperature stresses cannot be eliminated, although temperature changes undoubtedly have less effect on unit construction than on monolothic, on the same principle that a brick wall (which is constructed of small units) will generally show fewer cracks than a concrete wall. We have observed this action carefully and have had very little difficulty, if any, from temperature effects and account for it by the fact that during the construction of the building and before the connections are grouted, the units attain an average temperature.

Mr. Moore. MR. MOORE.—I take it therefore that it is necessary to waterproof most of this filling. Joints in factory floors are more especially referred to as these are washed down and there is objection to having the water go through the cracks.

Mr. Conzelman. MR. CONZELMAN.—The construction shown is absolutely waterproof so far as we know. A unit weighing 4 or 5 tons when set on a mortar bed will compress it to such an extent that it is practically watertight. One advantage of unit construction is that we know just where the joints are and as leakage is necessarily confined to the joints proper provision can be made.

Mr. Kinney. MR. WILLIAM M. KINNEY.—It would seem there would certainly be some little unevenness, which is one of the greatest troubles of the concrete floor and one of the things we are trying to eliminate in floors that have to be trucked over. The trouble occurs where the joints are made in a floor and the question arises whether in unit construction any protection is afforded over the joints.

MR. DARNELL.—There is no apparent joint at the edge of the floor slabs. The floor slab and beam are cast in one and the edges of the slabs are made so that the space between is in the form of a wedge, which is filled up level with mortar so that practically there is no joint left at all and the floor is perfectly smooth. That is perhaps one of the advantages of this form of construction. The edge of the floor slab is built to exact dimensions and if any unevenness develops it is very easy in setting the slab on the girder to level it up smoothly, which is done. If the levelling is carefully done and these wedge shaped joints filled up with grout, it makes the floor, to all intents and purposes so far as the surface is concerned, perfectly smooth. There has been no trouble with uneven joints in trucking over floors in any of the buildings. The National Lead Company's buildings in St. Louis, is a group in which trucking is constantly going on and there has never been a complaint at any time. Mr. Darnell.

MR. ALLEN BRETT.—The tests reported by the Committee on Reinforced Concrete and Building Laws, in the case of the beam and slab show that the concrete in the slab acts in compression practically all the way across. This girder used in unit construction is a broad inverted T, practically, and the slabs are inverted boxes, the edges of which rest on the flange of the T. Is there any compression acting with the girder in the slab? Mr. Brett.

MR. CONZELMAN.—The girders are made with ledges or shelves on each side upon which rest the floor slabs, usually made to resemble the cover of a large box. They consist essentially of a thin plate or slab carried by beams on each side, these beams returning around the ends of the slab; the arrangement gives the slabs a continuous or uniform bearing on the girder ledge. The thickness of the slab or plate may vary from 1½ to 5 or 6 in. depending on the load to be carried and the span between beams. When the slabs are placed the top of the slab is higher than the top of the girder, an amount equal to the thickness of the slab, and the steel projecting from the slabs on each side of the girder interlock in the space so formed; the stirrups from the girder also extend into this space. The joints between the girder and slabs are then grouted and the space filled with a rich concrete. After the grout has hardened these become for all practical purposes T beam sections. Mr. Conzelman.

Mr. Conzelman. This T-beam action has been demonstrated in the laboratory on large beams by measuring the deformation in the concrete directly over the girder and in the adjoining floor slabs and also the deformation of the reinforcement, by means of an extensometer similar to that which has been described by the Committee on Reinforced Concrete.

Mr. H. P. Green. MR. HERBERT P. GREEN.—What provision is made to take up the shear between the columns other than the brackets on the columns?

Mr. Conzelman. MR. CONZELMAN.—The brackets are designed to take the entire load from the girder; the brackets are designed for vertical shear and bending moment. The size of the top of the bracket is determined by the bearing area required to properly distribute the load from the girder to the bracket.

REPORT OF COMMITTEE ON SPECIFICATIONS AND METHODS OF TESTS FOR CONCRETE MATERIALS.

In this first report of the Committee no definite recommendations are presented for specific tests or methods of making tests of aggregate, except that the Committee recommends for a practical test of sand a determination of the strength of mortar made with it. The report, therefore, is in the nature of a tentative discussion of various tests and of methods that have been used in different laboratories. This progress report should be followed during the coming year by further investigations leading to more definite recommendations.

The Committee request that information on methods of making tests of aggregates and results obtained by such methods be forwarded to the chairman of the committee. Laboratories that are in a position to assist the Committee or undertake research work in the line of concrete aggregates are also asked to correspond with the Committee.

Importance of Testing the Aggregates.

The selection of aggregates for concrete, especially the selection of sand or fine aggregate, is of as great importance as the selection of the cement. So evident is this to the engineer who has had experience both in practical construction and in laboratory tests, that it is almost inconceivable that so much important work should be undertaken and carried through without testing the sand. Frequently every carload of cement is carefully sampled, but no test whatever, except by inspection, is made of the equally important ingredient, sand. When tests are made, they frequently are confined to mechanical analysis or granulometric composition. While this is a valuable test for comparing the qualities of different fine aggregates that other tests have shown to contain no impurities, it does not show up some of the worst defects that occur occasionally. It therefore cannot be relied upon alone.

The impossibility of determining the true quality of a sand or other fine aggregates by mere inspection cannot be emphasized too strongly.

SAMPLING AGGREGATES.

Samples must be taken in such a way as to obtain a fair average of the material to be tested.

The size of the sample depends upon the character of the aggregate and the nature of the test. One should err on the side of getting too large a sample rather than one that is too small. For tensile or compressive tests of mortar made with fine aggregate a sample not less than 20 lbs. in weight should be taken in order to have enough material left over for other laboratory tests that may be deemed necessary. If practical tests of proportions with coarse aggregates are to be made, the sample of fine aggregate should be several times larger than this.

The coarse aggregate sample should be larger than that of the fine aggregate in order to get fair average of the material, because the grains are larger and there is more variation in them. Whatever tests are made must be on a larger scale. For tests involving both mechanical analysis and volumetric tests of concrete mixtures for proportioning the aggregates, at least 200 lbs. of each coarse aggregate are needed.

Samples should be shipped in a strong box or a bag. It is advisable also that the natural moisture be retained as far as possible, so that the laboratory will receive the material in its natural condition.

For sub-dividing the sample to obtain the required amount for each test, different methods are employed in different laboratories. One of the common methods is that of quartering.

Quartering.—To quarter a sample of aggregate, it is spread out on a thoroughly clean floor or table, or else upon a large sheet of manilla paper. Care is used in spreading to see that particles of different size are distributed through the mass. The pile is preferably in the shape of a circular disc. The material in this shape is divided into four quarters. Two opposite quarters are removed, taking care to remove all dust. The remaining quarters are then mixed together. After mixing, the material is spread out again, as before, and quartered again. This process is followed until the quantity remaining is of the size required for the experiment.

TENSILE OR COMPRESSIVE TEST OF MORTAR.

The Committee recommends the test of the strength of mortar as that best indicating the quality of the fine aggregate.

To eliminate variations in result, due to the character of the cement, the difference in laboratory conditions, and the personal equation of the operator, the test always must be a comparative one. For comparison, standard Ottawa sand which is used for cement tests, and which every well-equipped laboratory should be provided with, is recommended.

The Joint Committee on Concrete and Reinforced Concrete in their 1908 report, makes the following recommendation for this test of strength:

> Mortars composed of one part Portland cement and three parts fine aggregate by weight when made into briquets should show a tensile strength of at least seventy (70) per cent. of the strength of 1:3 mortar of the same consistency made with the same cement and standard Ottawa sand.

While this requirement is far in advance of usual practice, where no laboratory tests are required, it is not so severe as should now be demanded in the present state of the art of reinforced concrete construction. For the present, the Committee recommends that sand used in reinforced concrete be accepted only after tensile or compressive tests of 1 : 3 mortar made with the sand in question, in comparison with similar tests of mortar of standard sand made up at the same time under the same conditions.

The Committee is not yet prepared to recommend a fixed value for the ratio of strength for acceptance. It is suggested for the present that the ratio be set to suit individual requirements.

To avoid the removal of any coating on the grains which may affect the strength, bank sand should not be dried before being made into mortar, but should contain natural moisture. The percentage of moisture to use for correcting the weights in measuring the proportions may be determined upon a separate sample. From 10 to 40 per cent. more water may be required in mixing bank sands or artificial aggregates than for standard Ottawa sand to produce the same consistency.

In the mortar tests, enough test pieces should be made to test at 72 hours, 7 days and 28 days, the first 24 hours all being stored in moist air, maintained at a temperature of 70 deg. Fahr.,

and the remainder of the period in water at the same temperature. The 72-hour test is the most severe, and sand failing to attain this requirement frequently reaches it at 7 days or at 28 days, and can then be accepted. If, however, the 72-hour briquets break in the clips of the machine, or if the test pieces at this age show very low strength, say 25 per cent. or less, of the strength of standard sand mortar, the sand should be considered dangerous to use on any important work of construction.

MECHANICAL ANALYSIS.

For proportioning fine and coarse aggregate, the tests of mechanical analysis are important. Curves of mixtures in different proportions, based on the combined analyses of the cement, fine aggregate, and coarse aggregate, may be drawn and studied to obtain the proportions corresponding most nearly to the ideal requirements. The proportions thus found may then be used in tests of volume, as referred to below.

Mechanical analysis of fine aggregate is valuable as furnishing an indication of its quality. It is recommended that fineness requirements be introduced into concrete specifications. This test cannot be relied upon fully, however, since there may be impurities in the sand that will make it unfit for use even when the analysis is satisfactory. The chemical and also the mineralogical composition of the sand also may affect its strength. Leaving out of consideration, however, the question of impurities, for which specific tests probably will soon be evolved, the test of mechanical analysis, or granulometric composition, as it is sometimes termed, is worthy of much further development. Laws which govern the effect of the sizes of the particles of the aggregate upon the resulting mortar or cement are not yet clearly formulated.

Further studies are necessary for the selection of standard sieves for use in mechanical analysis. From the report of the Committee on Reinforced Concrete of this Association,* the following paragraphs are quoted:

19. The relative strength of mortars from different sands is largely affected by the size of the grains. A coarse sand gives a stronger mortar than a fine one, and generally a gradation of grains from fine to coarse is

* See *Proceedings*, Vol. V, p. 457. — ED.

advantageous. If a sand is so fine that more than 10 per cent. of the total dry weight passes a No. 100 sieve, that is, a sieve having 100 meshes to the linear inch, or if more than 35 per cent. of the total dry weight passes a sieve having 50 meshes per linear inch, it should be rejected or used with a large excess of cement.

20. For the purpose of comparing the quality of different sands a test of the mechanical analysis or granulometric composition is recommended, although this should not be substituted for the strength test. The percentages of the total weight passing each sieve should be recorded. For this test the following sieves are recommended:*

0.250 inch diameter holes.†
No. 8 mesh, holes 0.0955 inch width No. 23 wire.
No. 20 " " 0.0335 " " No. 28 "
No. 50 " " 0.0110 " " No. 35 "
No. 100 " " 0.0055 " " No. 40 "

21. The effect of mechanical analysis or granulometric composition upon the strength of mortar is illustrated in Appendix. By this table (which follows) the relative strength of different sands may be approximately estimated.

TESTS BY NEW YORK BOARD OF WATER SUPPLY OF 1:3 MORTAR MADE WITH SANDS OF DIFFERENT MECHANICAL ANALYSIS.

Percentages Passing Sieves.				Tensile Tests.		Compression Test.	
No. 4.	No. 8.	No. 50.	No. 100.	7 days.	90 days.	7 days.	90 days.
100	70	12	5	213	613	2690	5640
100	86	21	6	263	412	1915	4660
100	99	26	2	177	325	905	2170
100	97	28	6	178	282	1070	1500
100	94	44	12	139	228	905	1130
100	100	52	14	122	170	275	810
100	100	94	48	80	149	330	490

VOID TESTS.

Void tests of coarse aggregates frequently are used to determine proportions for concrete. They do not give entirely correct results, however, unless the tests are made with mixtures

* Sheet brass perforated with round holes passes the material more quickly than square holes. Round holes corresponding to sieves No. 8, 20 and 50 respectively are approximately 0.125, 0.050, 0.020 inch diameter.

† A No. 4 sieve, having 4 meshes per linear inch, passes approximately the same size grains as a sieve with 0.25 in. diameter holes.

of all the ingredients. Usually some of the grains of the fine aggregate are so coarse as to force apart the grains of the coarse aggregate. If the sand is fine in proportion to the stone, less mortar will fill the voids of the stone than if the sand is coarser and therefore more nearly the size of the stone particles.

Void tests of fine aggregate are also affected by the fact that the cement forces the grains of sand apart. The voids in fine aggregate also are affected to a large degree by the percentage of moisture contained in it. If perfectly dry, a fine aggregate with grains of uniform size may have nearly the same percentage of voids as a coarse aggregate of uniform size grains, although the former will produce a very weak mortar and the latter a strong one. If the voids were based on the volume of the moist aggregate, the results would be more normal, but a slight variation in the percentage of moisture produces such a marked effect that it is impossible to make true comparisons in this way.

A common method of determining the voids in an aggregate is to place it in a measure, either loose or compacted as desired, and measure the quantity of water which can be poured in. The percentage of voids, either by weight or volume, is thus found directly. This method with a clean coarse aggregate is fairly satisfactory. With a fine aggregate, however, air is entrained and it is almost impossible to obtain correct results.

Another plan sometimes followed is to measure the material; place a definite quantity of water in a graduated vessel; pour the aggregate into the water; and determine from the graduated scale the difference in volume of the water before and after adding the aggregate. This difference is the amount of water which the aggregate displaces. This is substantially a specific gravity method.

A still simpler plan, if the specific gravity of the aggregate is known or can be readily determined, is to weigh a given bulk of the aggregate, loose or compacted as required, correct for moisture, and compute the voids directly.

WEIGHT.

Weight tests have the same limitation as void tests, since the weight is affected by the percentage of moisture contained in the aggregate. The weight also varies directly with the specific

gravity. If the specific gravity of the material is known and the percentage of moisture is determined, the voids can be computed, as indicated above.

VOLUMETRIC TESTS OF MORTAR OF FINE AGGREGATES.

If there are no organic, or other similar impurities, that abnormally reduce the strength, the aggregate producing, with the cement and mortar, the smallest volume of mortar or concrete is apt to give the greatest strength.

A volumetric test is better than an ordinary void or a weight test because the aggregate is mixed with cement and water as in practice.

The general method employed in making a test of volume is to mix up the aggregates in the given proportions by weight, add enough water to produce a consistency slightly softer than used in tensile tests, and determine the bulk of mortar or concrete made with this mixture. Knowing the specific gravity of all the materials used, the absolute volumes and density can be computed. The method of making this test is described more fully in a paper on Concrete Aggregates presented in 1906.*

VOLUMETRIC TESTS OF CONCRETE INGREDIENTS FOR DETERMINING PROPORTIONS.

One of the most valuable functions for volumetric tests lies in determining the proportions of concrete. The value of the test is based on the principle that, with the same proportion of cement, the mixture which gives the smallest volume, and is therefore the densest, usually produces the strongest concrete. This rule is not strictly true for permeability because the size of the voids as well as the density influence the permeability.

For determining the density of the concrete, the specific gravity of each aggregate must be known. The specific gravity of cement may be assumed as 3.10. An average specific gravity for bank sand is 2.65.

The process of making volumetric tests of mixtures of coarse and fine aggregates with cement is similar in principle to the volumetric test of mortar described. Larger volumes must be

* See *Proceedings*, Vol. II, p. 27.—ED.

used and it is sometimes easier to fill a measure of given size and determine the amount left over than to determine the bulk of the total material. A blank form for use in this test is referred to below.

MICROSCOPICAL EXAMINATION.

The mineralogical composition of a fine aggregate may frequently be determined approximately by an examination under a microscope or magnifying glass of high power. Quartz can be recognized and if dirt surrounds and adheres to the grains it can be seen.

CHEMICAL ANALYSIS.

The value of chemical analysis is not clearly defined. A quartz sand in general is better than a natural sand of other composition, chiefly because it is cleaner. It is claimed by some that the amount of clay material in the aggregate affects the sand in other ways besides increasing the amount of fine material.

The effect of colloids and colloidal action remains to be studied.

The ignition test referred to below is really a chemical test.

TEST FOR ORGANIC MATTER.

Experience with defective concrete indicates that the quality of a sand may be very poor through a minute quantity of organic or vegetable matter contained in it. The best method of testing for this and the limitations which must be placed upon the quantity are not yet clearly defined.

A method of test was suggested several years ago by the Chairman of the Committee and has been used in practice by him and also, more recently with some modifications, by another member of the Committee, Mr. Chapman. The methods are substantially as follows:

Two hundred grams of the damp sand just as it is received at the laboratory are weighed out and put into a jar or a graduate. If desired, the quantity of this may be measured and a given bulk, such as 100 c. c., may be used instead of the fixed weight. If thus measured by volume the weight is also determined. Water is added and the mixture is shaken violently and stirred for 2 minutes. Then the dirty water is poured off into a separate

vessel. More water is added and the operation is repeated until the water is practically clear. The water poured off is evaporated, taking care not to raise the temperature much above the boiling point, or else by another method, it is poured through a filter paper (previously weighed) and this residue, together with the paper, dried at a temperature of 212 deg. Fahr. The filter paper and residue are then weighed together, the weight of the filter paper deducted, and the remainder is considered as silt. The percentage is recorded which the weight of this silt, or, by the other method, the weight of the silt left from the evaporated water, bears to the weight of the original sand.

The evaporated residue, or else the filter paper with its residue, are ignited in a crucible at a red heat, and the loss of weight by this process (after allowing for the weight of the filter paper) is taken to indicate the amount of organic or vegetable matter. The percentage of this is expressed both in terms of the silt and of the total sand.

BLANK FORMS FOR REPORTS.

Forms are appended to this report that have been used in laboratories of members of your Committee for special tests of aggregates, see Figs. 1–4.

Respectfully submitted,

SANFORD E. THOMPSON, *Chairman*,
CLOYD M. CHAPMAN,
WILLIAM B. FULLER,
RUSSELL S. GREENMAN,
ARTHUR N. TALBOT.

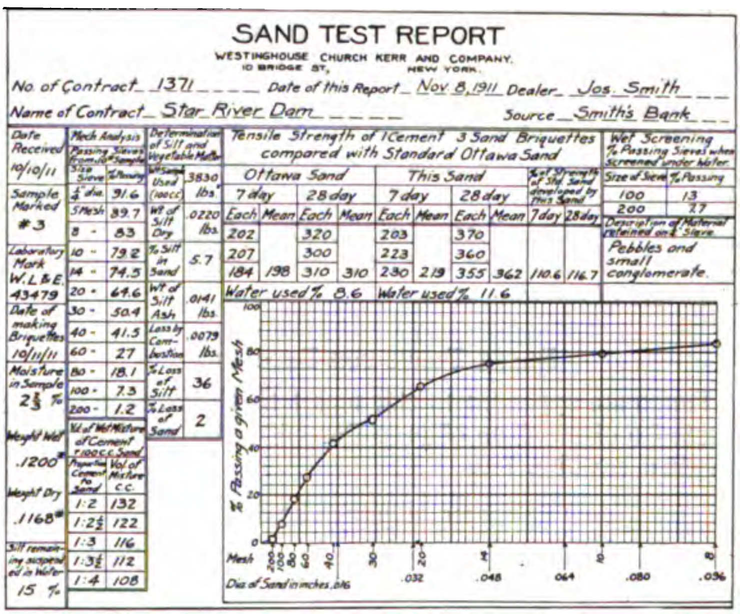

FIG. 1.—SAND TEST REPORT USED BY WESTINGHOUSE, CHURCH, KERR AND COMPANY. CLOYD M. CHAPMAN, ENGINEER IN CHARGE, NEW YORK, N. Y.

CONTRACT NO.

TESTS OF SAND from..............................bank at................................, N. Y.
Proposed for use on contract No........, Res. No........Canal,........Division.....
Contract sample No........taken........, received at Laboratory........, made up........
Sand is *mainly quartz and feldspar with some hornblend and magnetite.*
Percentage of Voids, *35.9;* Loam, *3.2;* Organic matter, *trace.*
Parts of sand to cement, by bulk:—*3* sand to 1 cement.
Per cent water used *+13.*
Cement used in tests ——————.
For test of cement see Vol. , Page .
Temperature, Fah. when mixed, Air , Water .
Briquettes kept in air 24 hours and then immersed.

TENSILE STRENGTH (in pounds per square inch)						SIZE OF SAND			
NATURAL SAND			WASHED SAND			PASSING SIEVE			
Briquette No.	7 days	28 days	Briquette No.	7 days	28 days	No.	Per cent.	No.	Per cent.
....	*178*	*252*	....	*182*	*286*	2	*100.0*	40	*33.2*
....	*186*	*274*	....	*196*	*276*	4	*100.T*	60	*15.0*
....	*192*	*250*	....	*190*	*254*	6($\frac{1}{8}$)″	*99.2*	74	*7.8*
....	*192*	*258*	....	*193*	*262*	10	*97.6*	100	*3.2*
....	*170*	*280*	....	*196*	*274*	20	*87.0*	140	*2.0*
....	......	......	....	......	......	30	*58.0*	200	*1.4*
Total	*918*	*1294*	Total	*962*	*1352*				
Average	*184*	*259*	Average	*192*	*270*				

Reported 19 .
Examined and Approved

Resident Engineer.
Accepted . Rejected.

Tests for strength made by .
Tests of sand made by . Recorded by .

FIG. 2.—REPORT CARD OF SAND TEST NEW YORK STATE TESTING LABORATORY, ALBANY, N. Y., RUSSELL S. GREENMAN, RESIDENT ENGINEER IN CHARGE OF TESTS.

SANFORD E. THOMPSON, CONSULTING ENGINEER, Newton Highlands, Mass.	Volumetric or Density Test.	File, *SWBE*. Date, *Feb. 18, 1911*. Experimentor, *W. O. L*

(1)	Test Number	*V-239*
(2)	Date	*2/18/11*
(3)	Nominal mix	*By volume* 1:2:4 *By weight* 1:1.8:3.8
(4)	Brand of cement	———
(5)	Weight of cement	*2.000*
(6)	Weight of aggregate passing a No. 100 sieve	*0.0007*
(7)	" " " coarser than a No. 100 sieve	*11.180*
(8)	" " vessel and water (before using)	*1.500*
(9)	" " " " " (after using)	*0.740*
(10)	" " water used	*0.760*
(11)	Total weight mixed (5)+(6)+(7)+(10)	*13.947*
(12)	Weight of tray+tools (after mixing)	*2.750*
(13)	" " " + " (before mixing)	*2.625*
(14)	Weight of mixed adhering	*0.125*
(15)	Weight of waste+water	*0.125*
(16)	Weight of waste	*0.094*
(17)	Weight of free water	*0.031*
(18)	Net mix of set = (11) − (14) − (17)	*13.791*
(19)	Water left on tray $\frac{(10)\times(14)}{(5)+(6)+(10)}$	*0.0343*
(20)	Net water set = (10) − (17) − (19)	*0.695*
(21)	Net cement (5) $-\frac{(5)\times(14)}{(5)+(6)+(10)}$	*1.9098*
(22)	Net aggregate passing a No. 100 Sieve = (6) $-\frac{(6)\times(14)}{(5)+(6)+(10)}$	*0.0067*
(23)	Depth of concrete in cylinder	*0.4427*
(24)	Volume of concrete in cylinder	*0.0869*
(25)	Net water per cu. ft. as mixed (10) ÷ (24)	*8.750*
(26)	" " " " set (20) ÷ (24)	*8.020*
(27)	Net cement per cu. ft. as set (21) ÷ (24)	*22.000*
(28)	Net aggregate per cu. ft. as set (22)+[(7) ÷ (24)]	*128.50*
(29)	Abs. vol. of water per cu. ft. as set (26) ÷ *62.3*	*0.1285*
(30)	Abs. vol. of cement (27) ÷ *62.3*	*0.1139*
(31)	Abs. vol. of aggregate (28) ÷ *62.3*	*0.760*
(32)	Abs. vol. total = (29)+(30)+(31)	*1.0024*
(33)	Weight of form +concrete	*19.000*
(34)	" " "	*5.156*
(35)	" " concrete	*13.844*
(36)	Temperature of water	*70°F.*
(37)	Time of mixing	*10.00 a.m.*
(38)	Remarks on consistency	*Jelly like*

FIG. 3.—MECHANICAL ANALYSIS REPORT, SANFORD E. THOMPSON, CONSULTING ENGINEER, NEWTON HIGHLANDS, MASS.

SANFORD E. THOMPSON,
CONSULTING ENGINEER,
Newton Highlands, Mass.

Mechanical Analysis.

File, *SWBE.*
Date, *Jan. 24, 1911.*
Experimenter, *W. E. S.*

Test made for..................*SWBE.*

Description....................*Gravel from Neponset River.*

Samples Taken (date)...........*Jan. 23, 1911. Shipped to laboratory in bag.*
Samples Received (date).........*Jan. 24, 1911.*
Weight Total Sample...........*20 lbs.*
Client's Mark
Laboratory Mark...............*A. S., 186 SWBE.*
Analysis No....................*A. S. 186.*
Per cent. Moisture.............*1.5%*

Size of Sieve		Total	Total	Per cent.	Total	Total	Per cent.	Total	Total
Inches	No.	Weight Passing	Per cent. Passing	Finer than ¼″					
2.50	2½	....	....	....	....	....	....	....	....
2.00	2	....	....	....	....	....	....	....	....
1.50	1½	*998.0*	*100.0%*	....	....	....	....	....	....
1.00	1	*677.2*	*67.8*	....	....	....	....	....	....
0.50	½	*569.0*	*57.1*	....	....	....	....	....	....
0.25	¼	*388.0*	*38.9*	*100.0%*	....	....	....	....	....
0.16	5	*365.0*	*36.6*	*94.3*	....	....	....	....	....
0.0583	12	*150.6*	*15.1*	*38.8*	....	....	....	....	....
0.0335	20	*64.2*	*6.4*	*16.5*	....	....	....	....	....
0.0148	40	*17.0*	*1.7*	*4.4*	....	....	....	....	....
0.0110	50	*11.8*	*1.2*	*3.1*	....	....	....	....	....
0.0055	100	*5.8*	*0.6*	*1.5*	....	....	....	....	....
0.0030	200	*3.5*	*0.35*	*0.9*	....	....	....	....	....

Remarks

		Date	Initials
Washing No. *W225*			
Tensile No. *T926*	Approved,	*Jan. 24, 1911.*	*W. O. L.*
Report made *Feb. 1, 1911.*	Noted by S.E.T.,	*Jan. 28, 1911.*	*S. E. T.*

FIG. 4.—VOLUMETRIC OR DENSITY REPORT.

AGGREGATES FOR CONCRETE.

By William M. Kinney.*

Within the past few years has been seen the almost universal adoption of the Standard Specifications for Portland Cement† of the American Society for Testing Materials. Practically all cement made in this country is guaranteed to meet these requirements and it is only occasionally that a shipment fails to pass the specifications by a safe margin. Despite the precautions taken in the making and testing of cement, however, there are occasionally complete or partial failures of concrete work. Fortunately these failures usually occur during the process of construction so that the loss of life and property is relatively small, but that there should be any loss whatever in this type of construction should lead such bodies as this Association to strive for greater efficiency by a careful and thorough study of the materials entering into and the workmanship required for concrete.

It is seldom possible to determine positively the cause for such failures; in fact, the number of reasons given for a particular failure usually varies with the number of engineers employed on the investigation. It is essential, therefore, that in the construction of any concrete or reinforced concrete structure, that each step be made with absolute surety in order to attain success.

Deficient strength in concrete is usually due to one or more of the following causes: poor workmanship, unsatisfactory aggregate and unfavorable weather conditions, assuming of course that the design is right and that the cement had been carefully tested to the standard specifications. Poor workmanship can be eliminated from this discussion, because with the widespread distribution of literature on the mixing and placing of concrete, it should be possible for any contractor or user to handle concrete in such a manner that success will be assured. However, the question of aggregate and temperature conditions, with usually a combination of the two, is a matter deserving of very careful consideration. Of these two, the question of aggregates is the

* Assistant Inspecting Engineer, Universal Portland Cement Company, Pittsburgh, Pa.
† Standard No. 1, National Association of Cement Users.—Ed.

paramount issue, as it is almost a certainty that if good, clean aggregates were used, at least three-fourths of the frozen concrete would be eliminated. The reason for making this statement is based upon the fact that when concrete is subjected to actual freezing weather during the early stages of hardening, it is usually well protected and poor work seldom results; but when it is deposited at a temperature above freezing it is not protected, and owing to delay in hardening due to unsatisfactory aggregates, freezing may occur the following night or even a number of days later. It is certain that in a great many cases had clean aggregates been used the concrete would have hardened sufficiently during the favorable weather to have withstood the subsequent freezing without resultant injury to the concrete.

The aggregates, therefore, play an important, if not the most important part in concrete work, and are without doubt subject to the greatest variation, yet have received up until the present time the least study of all the adjuncts of good concrete. To be sure, there are the studies of Feret, Candlot and other European experts. Likewise, the Structural Material Division of the United States Geological Survey (now under the Bureau of Standards) and individual investigators, such as Thompson, Spackman and Greenman have given valuable information obtained from their study of sand, gravel and other aggregates. This work, however, has of necessity been limited somewhat to the study of deposits local to the various laboratories, and the results are chiefly valuable in giving an idea of the most advantageous tests for detecting the properties of a particular aggregate, which render it good or poor for concrete. It remains to apply these tests in the study of the aggregates being used or proposed for use in individual structures or in particular localities.

The report of the Committee on Specifications and Methods of Tests for Concrete Materials has been presented. It would seem highly proper for this committee at the earliest possible date to confer the committees on Concrete and Reinforced Concrete of the American Society for Testing Materials, American Society of Civil Engineers, and Association of American Portland Cement Manufacturers, looking toward the establishment of a standard set of tests for concrete materials. Having outlined such methods, it should then be within the power of this committee to

influence the various Government, State and commercial bodies interested in this subject to inaugurate at once an extensive study of these materials by the methods outlined.

The magnitude of such an investigation may lead to the thought that it is not feasible, but there is no reason why, if properly organized under an advisory board composed of representatives of the various large engineering societies, the work cannot be systematically done without duplication of tests by the Bureau of Standards and the various State Universities and Experiment Stations together with whatever assistance may be rendered by other laboratories of a public or private nature.

In 1905 there was established at St. Louis a laboratory under the Structural Materials Division of the United States Geological Survey, which published in 1908 Bulletin 331 on *Portland Cement Mortars and Their Constituent Materials.* Neither this laboratory nor its successor in this line of work, the Bureau of Standards, has given any further data on this very important subject. It is to be hoped that the Bureau of Standards will continue this very necessary work, as it would be unfortunate to lose the practical information gained by the investigators in conducting the first series of tests. Good work has also been done in several of the State Universities and Experiment Stations. It is pleasing to note that the University of Wisconsin is to start at once under Mr. M. O. Withey, a comprehensive study of the aggregates of Wisconsin. If Wisconsin can do this, why cannot other States be interested in a similar investigation?

Such an investigation will not by any means solve the problem and insure good work in the future, but it will give an idea of the relative merits of the various aggregates available for concrete in any particular locality. This in itself will be a most important step forward, as it is safe to say that less than half of the architects and engineers in this country know the crushing strength that can be obtained from the mixtures they specify with the aggregates that are being used on jobs under their supervision every day.

This statement was recently borne out by an investigation of the aggregates available for concrete in one of our larger cities prior to the formation of a Building Code for concrete and reinforced concrete buildings. In this code it was originally planned

to have the requirement for compressive strength of 1:2:4 concrete at 28 days at least 2400 lb. per sq. in. This was later reduced to 2000 lb. and the allowable stresses figured accordingly. An investigation by the writer and others interested in the writing of the Code developed the fact that though the supply of sand and gravel was limited to practically one source and the crushed stone to several quarries in the same vein, very few tests on the aggregate were obtainable and these seemed to indicate that even 2000 lb. was too high. A series of tests was started and the results to date are quite surprising. The specifications

TABLE I.—COMPARATIVE COMPRESSIVE STRENGTH IN LB. PER SQ. IN. OF CONCRETE MADE FROM VARIOUS AGGREGATES.

Mark.	7 Days.	28 Days.	3 Mos.	Proportions.
Ordinary gravel..	677	1578	1512	1 part typical cement.
	596	1390	1673	2 parts river sand passing 3/16 in. screen.
	603	1633	1943	4 parts river gravel 3/16 in. to 3/4 in.
	625	1533	1709	
Large gravel....	898	1526	1684	1 part typical cement.
	677	1640	1779	2 parts river sand—3 parts through 3/16 in. screen, 1 part 3/16 in. to 1/4 in.
	874	1580	1649	4 parts river gravel 1/4 in to 1 in.
	816	1582	1704	
Stone..........	819	1824	2031	1 part typical cement.
	858	1972	2206	2 parts river sand—3 parts through 3/16 in. screen, 1 part 3/16 in. to 1/4 in.
	962	1880	2484	4 parts crushed stone 1/4 in. to 1 in.
	879	1888	2240	

Typical cement used was a mixture of five brands. Medium consistency. Hand mixed. Test pieces 8 in. in diameter and 16 in. long. Aged in temperature of about 70° F. and protected from drying out by cotton bags wet twice a day.

require that the coarse aggregate shall pass a 1-in. ring and be retained on a ¼-in. ring; fine aggregate to be all that passing a ¼-in. ring. The material produced commercially was being screened through a ¾-in. and over a $\frac{3}{16}$-in. mesh, so that tests were made on the aggregates as commercially produced and on samples specially prepared to meet the specifications. In the case of the sand, this was done by mixing with three parts of sand passing a $\frac{3}{16}$-in. screen, one part of the fine material passing a ¼-in. screen obtained from the $\frac{3}{16}$-in. to ¾-in. gravel. The results up to three months are shown in Table I and prove conclusively that with

TABLE II.—TESTS ON TYPICAL CEMENT USED IN COMPRESSION TESTS.

Fineness 100 mesh—94.8 per cent.
Fineness 200 mesh—77.4 per cent.
Soundness—Satisfactory.

Initial Set—4 Hours.
Final Set—7 Hours 25 Minutes.
Normal Consistency—24 per cent.

CHEMICAL ANALYSIS.

Silica	20.72
Alumina	7.24
Iron Oxide	2.84
Calcium Oxide	62.85
Magnesia	2.47
Sulphuric Anhydride	1.42
Moisture and undetermined	2.46

TENSILE STRENGTH IN LB. PER SQ. IN.

Neat.				1 : 3 Ottawa Sand.			
24 Hours.	7 Days.	28 Days.	3 Months.	3 Days.	7 Days.	28 Days.	3 Months.
310	600	740	765	180	340	420	440
315	660	780	775	190	300	390	480
330	660	750	740	200	300	400	460
318	640	757	760	190	313	403	460

1 : 3 Commercial Sand.			1 : 3 Special Sand. 3 parts through $^3/_{16}$ in., 1 part $^3/_{16}$ in. to $^1/_4$ in.		
7 Days.	28 Days.	3 Months.	7 Days.	28 Days.	3 Months.
240	390	390	290	460	505
300	380	425	340	420	480
265	365	380	290	400	535
268	378	398	307	427	506

GRANULOMETRIC ANALYSIS.

	Commercial Sand.	Special Sand.
Per cent retained on No. 4 mesh	0	0
10 "	10.6	31.5
20 "	19.2	14.5
30 "	25.8	15.2
40 "	11.8	10.5
50 "	5.8	4.2
80 "	24.4	18.5
100 "	0.6	2.3
Through 100 "	1.4	3.2

such proportions a requirement of 2000 lb. per sq. in. is too high for the local material, and that either the amount of cement will have to be increased or the proportions readjusted so as to produce higher results. In these tests, which were made on 8-in. diameter cylinders 16 in. long, the concrete was accurately proportioned and mixed by hand and the test pieces were stored in an even temperature approximating 70° F., being protected from

TABLE III.—COMPRESSIVE STRENGTH IN LB. PER SQ. IN. OF CONCRETE CYLINDERS USING MATERIAL FROM SAME SOURCE AS TABLE I.

Brand of Cement.	10 Days.	30 Days.	3 Months.	6 Months.
A	466	893	1450	1003
	498	960	1290	1185
	600	828	1087	1131
	521	893	1275	1106
B	439	990	1182	1152
	539	904	1096	1063
	409	800	1018	1299
	462	898	1098	1171
C	520	820	1433	1333
	527	999	1172	949
	515	1142	1136	1464
	520	987	1247	1248
D	395	774	1138	1084
	521	872	1253	1382
	536	898	1316	1609
	484	848	1235	1358
E	430	706	1263	1080
	396	834	1219	1064
	434	1084	1160	1045
	420	878	1214	1063

Proportions: 1 part cement (5 brands used individually), 2½ parts river sand through $\frac{3}{16}$ in., 5 parts river gravel $\frac{3}{16}$ in. to 1½ in.

Test pieces 8 in. in diameter and 16 in. long, mixed with batch mixer, stored in open shed and sprinkled night and morning for first 7 days.

drying out by cotton bags wet twice a day. The cement used was a mixture of five representative brands, the results of tests on which are shown in Table II. In the same table are granulometric and tensile tests on the commercial and special sand. It will be noted that despite the fact that the commercial sand approximates very closely the tensile strength of Ottawa sand and the special sand exceeds in strength that of Ottawa sand, yet

used in concrete with gravel from the same source and a very good grade of crushed stone, neither of these materials developed a crushing strength of 2000 lb. per sq. in. at 28 days. Other

TABLE IV.—COMPARATIVE COLD WEATHER TESTS ON (8 IN. DIAMETER BY 16 IN. LONG) CYLINDERS MIXED 1 CEMENT, 2½ SAND, 5 GRAVEL, DECEMBER 4, 1911. CYLINDERS PROTECTED FROM FROST BY COTTON BAGS. HAND MIXED, MEDIUM CONSISTENCY.

Crushing Strength in lb. per sq. in.

Age.	Brand "A"	Brand "B"
3 days in air and 1 day in laboratory	116 122 119 Average	95 95 95 Average
7 days in air	227 265 246	199 209 205
14 days in air	484 458 471	440 388 414
30 days in air	640 739 689	495 653 574

TEMPERATURE—DEGREES FAHRENHEIT.

	Max.	Min.	Mean.		Max.	Min.	Mean.
1st day	30	18	24	16th day	34	28	31
2d "	36	21	28	17th "	41	24	32
3d "	48	27	38	18th "	49	36	42
4th "	54	29	42	19th "	52	44	48
5th "	54	40	47	20th "	47	33	40
6th "	54	48	51	21st "	39	33	36
7th "	66	46	56	22d "	45	34	40
8th "	61	58	60	23d "	56	40	48
9th "	59	45	52	24th "	57	23	40
10th "	51	35	43	25th "	23	14	18
11th "	42	32	37	26th "	32	18	25
12th "	47	39	43	27th "	45	32	38
13th "	58	38	48	28th "	54	28	41
14th "	38	32	35	29th "	29	26	28
15th "	33	30	32	30th "	34	23	28

results obtained on material from the same source are shown Tables III and IV, and the latter table particularly shows that this aggregate is poor for cold weather work. Great care should be exercised in the use of such material in the winter time as

unless the materials are heated and the concrete well protected, poor results are almost inevitable.

Having established the value of aggregates obtained from any particular source, it is then necessary to see that the materials received on the job are equal in quality to the samples tested. An examination of the bank, pit or quarry may often reveal the fact that it would be impossible to obtain a uniform product. In such cases, extreme care must be exercised by the men obtaining the material and frequent tests should be made to see that that received is right.

The pier shown in Fig. 1 is a good illustration of the effect of variation in aggregate. This pier was built in connection with two abutments to support a steel girder railroad bridge across a small stream. Prior to starting the concrete work the engineer secured a sample of the sand and gravel which the contractor proposed to use, and the laboratory results obtained indicated that the materials as sampled were entirely satisfactory for concrete work. That the material as received was not good, and apparently not equal to the sample, is evidenced by examination of the condition of the concrete. A photograph of the gravel bank (Fig. 2) reveals the seat of the trouble. The dark streaks are apparently decayed vegetable matter, while here and there through the bank will be noticed strata of very fine uniform size sand. Examination of such a bank indicates quite conclusively that the aggregate could not run uniformly, and undoubtedly the delay in hardening which finally resulted in freezing was due to the presence of too much fine sand and loam. That this was the cause of the trouble is conclusively proven by the fact that the poorest results are evidenced at the water level where the fine material held the moisture and at the end of a day's work about midway of the pier where most of the fine material floated to the top of the concrete.

On another job of a similar nature, but involving the use of approximately 50,000 barrels of cement, the results were analogous. A large amount of concrete was condemned on account of unsatisfactory strength. In the middle of a 1000-yd. foundation laitance was found over a foot thick which had the appearance of wet clay and could be readily dug out with a knife even though the concrete had been in a month. Run-of-pit gravel was

being used which had been tested for percentage of fine and coarse material and found satisfactory, but examination of the pit revealed the fact that it would have been impossible to have secured a sample which would have fairly represented the deposit.

FIG. 1.—DISINTEGRATED CONCRETE PIER, DUE TO POOR AGGREGATES.

In some places there was very little sand, while in others there was practically no gravel. As a mixture of one part cement and seven parts run-of-pit gravel was being used, the natural result of the use of the latter material can be imagined. Further than

this, there were strata of very fine sandy loam throughout the pit and the large section of laitance was the result.

The foregoing are a few of the many examples which can be cited where failure to appreciate the value of having good aggregate has led to trouble and such examples emphasize the importance of tests of the aggregates. It is quite surprising to find architects and engineers basing their designs on a certain strength of concrete which cannot be obtained or may be largely

FIG. 2.—GRAVEL BANK FROM WHICH POOR AGGREGATE WAS OBTAINED.

exceeded in actual practice, and surely our work is devoid of the fundamentals of good engineering when we use without discrimination aggregates of high and low strength giving values. A more thorough study of our sources of supply for concreting materials and a more careful inspection of these materials as they are received on the work, is of utmost importance to the industry and it should be one of the first efforts of this Association to study this important subject.

A partial bibliography of the literature on aggregates for con-

crete is given below with the hope of furthering study on the subject.

Concrete Aggregates, by Sanford E. Thompson, *Proceedings* National Association of Cement Users, Volume II, p. 27. See also *Engineering Record*, January 27, 1906.

Sands and Their Relation to Mortar and Concrete, by Henry S. Spackman and Robert W. Lesley. *Proceedings*, American Society for Testing Materials, Volume VIII, p. 429.

The Value of Sand in Concrete Construction, by E. S. Larned, *Proceedings*, National Association of Cement Users, Volume IV, 205.

Concrete—Its Constituent Materials, by Russell S. Greenman, *Barge Canal Bulletin* (New York State), November, 1909, p. 429.

Practical Tests of Sand and Gravel Proposed for Use in Concrete, by Russell S. Greenman, *Proceedings* American Society for Testing Materials, Volume XI, p. 515. See also *Engineering Record*, Volume LXIV, No. 3, p. 66.

Economical Selection and Proportion of Aggregates for Portland Cement Concrete, by Albert A. Moyer, *Engineering-Contracting*, Volume XXXIII, No. 3, p. 52.

Concrete Aggregates by Dr. J. S. Owens, *Concrete and Constructional Engineering*, March 1909.

Portland Cement Mortars and their Constituent Materials, by Richard L. Humphrey and William Jordan, Jr., *Bulletin* 331, United States Geological Survey.

Good Concrete and How to Get It, by F. M. Okey, *Municipal Engineering*, May, 1909.

Notes on Concrete, a discussion printed in *Journal* of Association of Engineering Societies. See *Engineering Record*, Volume LXI, No. 5, p. 125.

Impurities in Sand for Concrete, a discussion,—Transactions American Society of Civil Engineers, September, 1909.

Blast Furnace Slag in Concrete, booklet published by Carnegie Steel Company, Pittsburgh, Pa.

Tests to Determine Effect of Mica on Strength of Concrete, by W. N. Willis, *Engineering News*, February 6, 1908.

Directions and Suggestions for the Inspection of Concrete Materials, by Jerome Cochran, *Engineering-Contracting*, Volume XXXVII, No. 5, p. 115.

A Study of Sand for Use in Cement Mortar and Concrete, by E. S. Larned, *Journal* Association of Engineering Societies, Volume XLVIII, No. 4, April, 1912.

DISCUSSION.

PRESIDENT HUMPHREY.—The relation of the sand to the mortar, especially of the strength of mortars made of the commercial sand to that of mortars of standard Ottawa sand, is of interest. There has been a great deal of debate in the Joint Committee as to whether they should show the same strength or possibly less. Engineers from New England, especially from Boston, object to requiring the same strength, because in their locality it is impossible to obtain a sand that would show this strength, therefore making the requirement a hardship. I believe, however, with Mr. Kinney, taking sand the country over, that the various sands should show at least the strength of the Ottawa sand, and that localities where it is not possible to obtain this strength must fix the requirement to suit their locality. It is certainly a fact that the well graded sands give higher results than the standard Ottawa sand. The standard Ottawa is a one size sand with a large percentage of voids and the strength of the mortar in which it is used is less than that of sands well graded. Mr. Humphrey.

Mr. Kinney states that grading gives density and that the increased density gives an increased strength. The retaining walls with the disintegrating mortar referred to are as fine an example of cause and effect as could possibly be had. It did not require a technical man to look at the sand bank to see that the material was not suitable for mortar. It was quite evident that the fine sand in the mortar retarded the hardening, and left the mortar with insufficient strength and thus easily damaged by frost action. This is one of the most important subjects that we can discuss. There is more information desired and great need for intelligent understanding of just what important part sand plays in mortars and concrete; it probably has a more important bearing on the resulting strength of the structure than the strength of the cement. I think we are more prone to ascribe the defects

in mortar or concrete directly to the cement rather than the sand used.

Mr. Kinney. MR. WM. H. KINNEY.—The sand from this particular bank had been tested by the engineer and was found to have a strength which compared favorably with Ottawa sand. In fact, it passed the 70 per cent strength requirement by a very satisfactory margin. However, the samples did not fairly represent the material the contractor was obtaining from the bank.

Mr. Wilson. MR. PERCY H. WILSON.—There was a case called to my attention 5 or 6 months ago where a railroad engineer endeavored to obtain in the laboratory a compressive strength of 2,000 lb. per sq. in., the requirement of the Joint Committee. On obtaining only about 1,800 lb. the test was tried over and over, varying the aggregate, but the strength never went up to 2,000 lb. Now is this requirement of the Joint Committee too high?

Mr. Humphrey. PRESIDENT HUMPHREY.—Many contractors and probably a good many engineers and architects never seem to think it is necessary to test the sand. They simply pass on it from a visual inspection, whereas the committee states that a differentiation between a good or a poor sand can only be made by actual test.

I would say I do not think the standard requirement set by the Joint Committee of 2,000-lb. concrete is too high. It is a standard we should work to. Unfortunately there is the practice in this country of specifying the proportions such as 1 : 2 : 4 or 1 : 3 : 6. Such proportions mean absolutely nothing, as the proper portions of materials cannot be determined until the qualities of the material itself are known. The voids vary with all the material all over the United States. A study of Bulletin No. 331* on the subject of sand will show that there is a wide range of variation, first, as to the size of the particles, and second, as to the hardness and character of the material itself. A traprock with a high compressive strength used as an aggregate in a certain proportion will give a concrete of much higher strength than an aggregate of very soft limestone. If a contractor with a fixed proportion cannot obtain 2,000 lb. or with any variation of the proportions, then more cement must be used or another aggregate obtained. The mere fact that his material in standard proportions will not give 2,000 lb. in my mind does not mean that he

*U. S. Geological Survey.

should not try to get 2,000 lb. There are too many structures being erected in this country assumed to have a strength of 2,000 lb. and upwards where as a matter of fact the strength is very materially less. The sooner more attention is given to the strength of the concrete as well as of the cement and the steel, the better the results will be. Mr. Humphrey.

The Joint Committee report specifies that the relation of the cement to the aggregates shall be 1 to 6. The next report will contain a table showing the strength of concrete from aggregates of different character and proportions, so that for any aggregate, say a soft limestone, reference to the table will show approximately the proportion of cement and aggregate necessary to obtain a 2,000-lb. concrete.

MR. L. R. ASH.—In connection with the sand and grading of aggregates suggested by the disintegrated concrete in the pier, the water put into a mixture sometimes makes considerable difference and also the depositing of concrete in or through water. I want to call attention to a very interesting occurrence here in Kansas City a couple of months ago. We have a bridge over the Kaw River built some 4 or 5 years ago and lately during some very intensely cold weather, the cylinder pier split open and went down so badly that false work had to be placed to hold the bridge up. In the search for the cause the first suggestion was that the contractor had not put in any cement but some line that seemed to be accessible in the neighborhood. I was talking to the foreman of the job and he told me the circumstances, which it seems to me would explain it very clearly. The concrete was deposited through several feet of water with the result that there were layers, at intervals, of material more like laitance than anything else, which was chalky in its consistency and would absorb great quantities of water. To me this very thoroughly explained the breaking down of the concrete. I have frequently noticed in depositing concrete that small pockets or irregularities in the distribution of the water would create conditions that would show up badly, and entirely outside of any irregularities in the aggregate at all. Sometimes I think that is a matter which is overlooked in the handling of concrete and depositing of the same in the forms. Mr. Ash.

MR. KINNEY.—I would like to ask whether anyone has noticed in the fracture of gravel concrete that the pebbles pull out rather Mr. Kinney.

Mr. Kinney. than break? This was particularly noticeable even at six months in these building code tests and it occurred to me that it might be a characteristic of all gravel concrete in the early stages of hardening.

If anyone contemplates the making of test cylinders on a job or in the laboratory, I might add that we have found a very satisfactory form made of galvanized iron. This form consists of a flat piece of galvanized iron 16 in. wide and long enough to come together and be soldered to form an 8-in. cylinder. The bottom is circular and slightly larger in diameter than 8 in. and is soldered to the cylinder. With the use of a sharp instrument these soldered joints can be readily opened and the forms stripped. Such forms cost 22 cts. apiece and they are worth the price.

Mr. Humphrey. PRESIDENT HUMPHREY.—Whether or not tests of cylinders would show a fracture through the gravel itself, depends largely on the age and the proportions. A proportion of 1 : 3 : 6 probably up to a year might not show such a failure. In many tests made in St. Louis the gravel did fail, especially 1 : 2 : 4 mixtures. The fracture was right through the particle, even the hardest particles. The governing factor seems to be the manner in which the materials are handled; that is, the character of the concrete and its age. A well made gravel concrete of 1 : 2 : 4, or in which there is one part of cement to six parts of gravel, at the end of six months should certainly show a fracture through the gravel.

FIELD INSPECTION AND TESTING OF CONCRETE.

By G. H. Bayles.*

In the spring of 1910 we were commissioned to design and superintend the construction of a warehouse 100 x 210 ft., 4 stories high, in the Borough of Brooklyn, New York, for the New York Dock Company. The use of reinforced concrete was recommended and adopted, but from experience and knowledge of the many criticisms of this material and the considerable number of failures in its use, it was decided that very careful inspection should be maintained. Consequently when the work started in the latter part of July the engineer who designed the building was put in charge, assisted by two inspectors, both experts in the making and placing of reinforced concrete. The engineer assumed general charge of the work. One inspector superintended the proportioning and mixing of the concrete and the other the placing in the forms. They collaborated in the inspection of constructing and wrecking forms, placing reinforcement and testing materials.

The results of the tests on the above work were on the whole so satisfactory and the information obtained was deemed of such importance that they were continued on some other work during the summer and fall of 1911. This latter work consisted of the reconstruction of a block of six warehouses. The whole block is 210 x 374 ft. in plan and 4 stories high. The buildings were of the old type, brick walls with wood interior. All the wood was removed and replaced by reinforced concrete, making the building fireproof throughout. As this work was done with greater despatch than the first job it was thought best to have more inspectors. Consequently a chief inspector was put in charge of the whole work, assisted by one inspector on the proportioning and mixing, one on the placing concrete and one solely for making tests. This arrangement worked very satisfactorily and allowed the engineer to devote more of his time to other work.

There seemed to be no recognized system of testing concrete

* Assistant to J. W. Galbreath, Consulting Engineer, New York, N. Y.

or concrete materials on the job, so a series of tests was arranged to meet what were believed to be the necessary requirements. These tests were for cement: fineness, constancy of volume, time of setting, initial and final, specific gravity and crushing strength of 1: 2 cement mortar; for sand: tests for loam, fineness and percentage of voids; for stone: tests for percentage of voids; for concrete: tests of the crushing strength. No tests of the reinforcement were made on the job.

A temporary laboratory was built near the work and the necessary apparatus installed. This laboratory was intended to be such as could be built on any job and no arrangements were made for heating it except the use of a small oil stove in extreme weather. It was 12 x 8 ft., built of $\frac{7}{8}$-in. sheeting on 2 x 4 in. frame with felt roof; one door front, and windows one side and rear.

CEMENT TESTS.

At first 6 samples for testing were taken from every car of cement, but later this number was reduced to 3. For fineness the usual 100-mesh and 200-mesh sieves were used. The inspector shook the sieves in his hands and judged by the eye when no appreciable quantity of cement was going through. Scales graduated to milligrams were used to determine the proportions. A nearby tinsmith made the boiler and wire rack for the constancy of volume tests and the glass plates were cut on the job from scrap window glass. The boiler is 15 in. high and 8 in. in diameter with a lid, and the rack was made to hold 12 pats at a time. The test pats were boiled for 5 hours or more. The Vicat needle was used to determine the time of setting. The variable temperature made the results of these tests vary so greatly as to be of little value. Le Chatelier's specific gravity apparatus was used with gasoline for determining the specific gravity.

The test for crushing strength, not being in such general use as the other tests, requires more detailed description. It consisted in testing to failure 4-in. cubes of 1: 2 cement and sand mortar. More elaborate apparatus was necessary for making these tests. For this purpose were required gang molds for the cubes, a damp closet large enough to hold 4 gang molds, pans of water in which to submerge the test pieces, and a compression testing

machine. The gang molds, Fig. 1, were made of 1-in. lumber held together by iron clamps. The measurements were not exact, but the surface areas were approximately correct within the limits of accuracy of the tests. While only 2 cubes were used for each test the gang molds were made for 4 cubes, as the same molds were to be used for the concrete tests. They were made by a carpenter on the job and the clamps were made by the company's blacksmith.

The damp closet was built into the corner of the laboratory, the sides and floor of the laboratory forming two sides and the bottom of the closet; the other sides and top were of ⅞-in. tongued and grooved sheeting. Two shelves were put in the

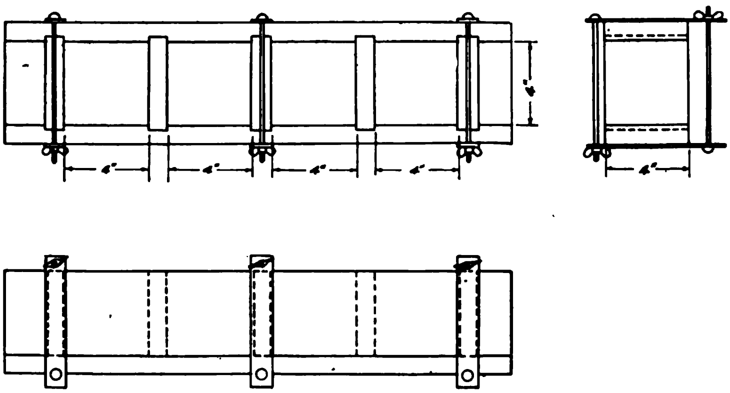

FIG. 1.—GANG MOLD FOR 4-IN. CUBES.

closet, each large enough to hold 2 gang molds, and space was left below for a large pan of water. When a mold with test pieces was placed in the closet a wet coffee sack was thrown over it, the ends of the sack dipping into the water below. A better arrangement would be to have the closet lined with felt, the lower edge of the felt dipping into the water. The simpler method was used as being one easily provided on any job.

The pans for water in which the test pieces were to be submerged were made 28 x 28 in. and 5 in. deep, large enough to hold 36 cubes each. The same size pan was used in the damp closet. Five pans were required and they were made by one of the company's sheet metal workers.

The selection of a compression testing machine was more difficult. Most machines designed to do this kind of work are elaborate and expensive. A simple and comparatively inexpensive machine was desired and after careful investigation a hydraulic machine was acquired, consisting essentially of two horizontal plane surfaces between which the test pieces are crushed. The top surface is fixed, being held rigidly in place by two side posts extending upward from the base. The lower or movable surface is fixed rigidly on top of a 5-in. cylindrical ram with cup leather packing moving in a copperlined cylinder. The pressure is applied by means of a hand pump and is measured on a gauge reading total tons on the 5-in. ram. The gauge is graduated from 0 to 60 t. and while the rated capacity of the press is 50 t., cubes were crushed which required as much as 60 t. pressure. As many of the newer cubes crushed at low pressure and exact readings on the gauge that was furnished on the machine were difficult, a second gauge was attached reading up to 10 t. only. When cubes requiring a greater pressure were being tested this second gauge was shut off by a valve.

On account of the high values which were obtained at the beginning of the work suspicion was directed towards the accuracy of the machine, but comparisons of the cubes by testing on an accurately gauged machine at the testing laboratory of Columbia University showed that the small press used on the work was correct within reasonable limits* (see Table IV).

For each test 2 cubes of 1: 2 cement and sand mortar were made. The sand used was taken from that used on the work and was washed clean. To insure uniformity a sufficient quantity for all the tests was taken at one time and stored in a bin until required. For each test a little in excess of 128 cu. in. of sand and half as much by volume of cement were used. The sand and cement were mixed dry, then sufficient water added to make a fairly wet mixture, that is, one that was readily puddled but would not pour. The whole was then well mixed and placed in the molds and puddled with a small trowel. All the operations were done by hand. As soon as the mortar was sufficiently set to hold the markings the date and hour were marked on both

* A description of a test of a similar machine is given in *Engineering News*, February 11, 1909, p. 167.

cubes and the mold placed in the damp closet. At the end of 24 hours the mold was taken from the damp closet, the forms removed and one cube crushed. The other cube was submerged in a pan of water where it remained for 6 days and was then crushed. All were tested to failure.

To prevent little inequalities of the surfaces of the cubes from seriously affecting the results three thicknesses of blotting paper were used top and bottom of each cube in the press. After some time it was discovered that the long sides of the gang molds were more nearly parallel than the others and thereafter care was taken to use the sides of the cubes formed by them as the crushing faces. By this means better and more nearly uniform results were obtained.

SAND TESTS.

Every scow load of sand was tested for percentage of loam. If the sand in different parts of the scow appeared to be of different quality more than one test was made. This test consisted in taking a quantity of sand and drying it thoroughly on a stove. A thousand grams of it was then taken in a 12-quart pail and washed by turning on a hose, giving the water as it flowed from the hose just sufficient velocity to keep the mass stirred up and moving, the loam being carried off in the overflow. When the water ran clear the sand was again dried and weighed, the difference in weight giving the percentage of loam. Sand containing more than 2 per cent was rejected.

Tests were also made to determine the comparative fineness of the sand. These were only made when there appeared to the eye to be a difference in the grade. The test consisted in taking a measured quantity of sand (by weight) and screening it by hand through 20-mesh and 30-mesh sieves. These tests showed on an average about 21 per cent retained on the 20-mesh sieve, 27 per cent on the 30-mesh sieve and the remainder passing through the 30.

Occasional tests for percentage of voids in the sand were made as follows: A comparatively large amount of sand was measured and weighed to show the average weight per cu. cm. Then a quantity by weight corresponding to 50 cu. cm. was put into a glass graduate and the volume of water displaced meas-

ured. The average volume of voids was thus found to be about 40 per cent.

STONE TESTS.

As the grade of the crushed stone varied but slightly only occasional tests were made to determine the percentage of voids in the stone. To do this a quantity of stone was submerged in water for 2 hours to permit it to become thoroughly saturated. The water was then poured off and the stone exposed to the air half an hour to permit the surface moisture to evaporate. A vessel of known capacity (nearly 10,000 cu. cm.) was then filled with the stone and weighed. Sufficient water was added to fill all the voids and the weight taken again. The difference in weight showed the percentage of voids. As there was excess of voids around the sides of the vessel this test is only comparative and is not considered particularly important as the inspector can best judge by the appearance of the batch when the voids are properly filled. For the stone used on this work the voids measured by the above method averaged nearly 50 per cent. Measurements of concrete in place and stone on the scows indicated that the volume of stone used was about 97.5 per cent that of the concrete produced.

CONCRETE TESTS.

What seemed to be the most important test and the test of most practical value was that of the crushing strength of the concrete itself as it was placed in the forms for the building. For these tests the same apparatus was used as for the mortar tests. The preparation of the test pieces was simpler. The gang mold for four 4-in. cubes was taken onto the work where concrete was being poured. The concrete was taken from the buggies as it was being poured into the forms and was immediately placed in the cube mold and puddled with a small trowel. There was no excessive puddling as it was intended to approximate working conditions as nearly as practicable. One and sometimes two tests were made every day concrete was poured. From this time on the operation was similar to that for testing mortar. After 24 hours in the damp closet the forms were removed, 1 cube crushed and the 3 others submerged in a pan of

water. Of these 3 one was crushed at the end of 7 days, 1 day in the damp closet and 6 in water, one at the end of 28 days and the other was removed from the water at the end of 28 days and laid aside to be tested later.

As has already been said, no attempt was made to preserve an even temperature in the laboratory, the object being to maintain as nearly as practicable the condition of the concrete in the work, so that it sometimes happened during the winter that ice formed on the water in the pans where the test cubes were submerged. The effect of this is clearly shown in the results of the tests. A practical use of the tests of concrete was to indicate to the inspector as well as the contractor the safe time for wrecking forms. This time was fixed at first at 7 days, but was afterwards shortened to 4 days during the summer and well into the fall, when the decreasing strength of the cubes tested caused the time to be again extended to 7 days.

RESULTS OF TESTS.

There is nothing notable in the results of the tests for fineness, constancy of volume and specific gravity, except that they show what may be expected from the ordinary inspector who is not specially trained in making such tests and where the tests are made under the conditions prevailing on the ordinary job. The results of the tests for initial and final set were so varied, due, no doubt, to the changing temperature, that they lack value as determining the quality of the cement.

Tests of the cubes of 1: 2 cement and sand mortar were intended to show a definite relation of strength of the cement being tested to what could be expected from the concrete made from the cement. Comparisons of the results of these tests with the results of the tests of concrete were so simple and proved so satisfactory that it is believed they show with sufficient accuracy the practical rate of setting of the cement and that the tests for initial and final set could be discontinued for field laboratories.

Strength of Mortar.—The 173 tests of 1: 2 cement and sand mortar on the first job showed an average compressive strength of 448 lb. per sq. in. in 24 hours and 2110 lb. per sq. in. in 7 days. On the second job, where a different brand of cement was used, 198 tests were made and the average compressive strength was

585 lb. per sq. in. in 24 hours and 2100 lb. per sq. in. in 7 days. Table I* shows in detail the results of the tests on the former and Table II those on the latter job. The results are recorded as found with the inspector's notes, the dates alone being omitted for lack of space. In Table I the first 50 tests were made between August 17 and October 3, 1910, the next 50 between October 3 and November 21, 1910, the next 50 between November 21, 1910, and January 25, 1911. The remaining 23 were made between January 25 and March 16, 1911. In Table II the first 50 tests were made between July 11 and August 24, the next 49 between August 24 and September 22, the next 50 between September 22 and October 21 and the remaining 49 between October 21 and November 25, all in 1911.

Strength of Concrete.—The 235 tests on concrete cubes on the first job showed an average compressive strength at 28 days of 3450 lb. per sq. in. and on the second job, 96 tests, 2321 lb. per sq. in. at 28 days. The tests on concrete cubes were carried on from August, 1910, to December, 1911, tests being made nearly every day except during the month of February, 1911, when no work was done on account of inclement weather, and from the completion of the first job in May, 1911, until the beginning of the second job in July following. During the latter part of December, 1910, and throughout January, 1911, the sand and water used in making the concrete were heated so that the batch had a temperature of 60 to 65 deg. when placed in the forms. Table III shows the complete record of the tests on concrete cubes on the first job, and Table IV those on the second job, both giving the inspector's notes of variations from the rule. While there are considerable variations of strength from the maximum to the minimum most of the cubes crushed near the average and reference to the final results indicates that the variations were due more to the method of testing than to the quality of the concrete, for while a cube crushed after 24 hours may have been below the average, the cube of the same set crushed after 28 days was about as often above the average as below it and vice versa. It seems evident that the actual strength of the concrete

* The inspector's notes give all the strength tests in tons per 4-in. cube, but for convenience of comparison with other tests the results have been converted into lb. per sq. in. in printing the tables.—ED.

TABLE I.—Tests of Cement and 1 : 2 Mortar.

Car Number.		Percentage of Cement. Retained on Sieve. 100 Mesh.	Percentage of Cement. Retained on Sieve. 200 Mesh.	Percentage of Cement. Through 200 Mesh.	Setting. Initial. Hrs.	Setting. Final. Hrs.	Specific Gravity	Crushing Strength of 4-in. cubes, in lb. per sq. in. 24 hrs.	Crushing Strength of 4-in. cubes, in lb. per sq. in. 7 days.	Boiling 5 hours.
A		11.250	23.800	64.860	1½	2½	3.1	500	....	
		....	....	....	....	....	....	563	2125	O. K.
		....	....	....	....	....	....	500	2315	....
B	1	7.150	20.075	72.100	2	3½	3.15	1053	2583	O. K.
	2	5.870	25.010	68.430	....	....	3.13	1250	3190	O. K.
	3	7.210	21.050	71.160	2½	4½	3.11	1000	2350	O. K.
	4	....	....	....	2¾	4	3.11	1188	3190	O. K.
	5	7.050	20.300	71.850	3	5¼	3.15	1125	3220	O. K.
C	1	8.260	20.630	70.620	....	....	3.07	750	1870	O. K.
	2	8.300	21.400	69.750	4	5	3.12	1250	2500	O. K.
	3	7.455	18.560	73.730	4	....	3.12	657	2290	O. K.
	4	7.400	23.310	71.250	5¾	7	3.13	500	2150	O. K.
	5	7.080	21.490	70.000	4¾	6¾	3.10	500	2350	O. K.
	6	8.020	22.520	68.770	4	6	3.12	845	2230	O. K.
D	1	6.980	22.140	70.490	3½	5½	3.15	875	2780	O. K.
	2	6.130	20.700	72.850	3¼	5	3.14	813	2370	O. K.
	3	6.300	21.520	72.000	3½	7	3.12	813	2275	O. K.
	4	6.070	23.140	70.300	4	7	3.12	375	1940	O. K.
	5	7.170	19.000	73.100	2½	3¼	3.11	500	2750	O. K.
E	1	6.830	20.100	73.050	3½	5	3.15	750	2650	O. K.
	2	7.050	25.020	67.830	3¾	6¼	3.15	750	2400	O. K.
	3	6.790	22.940	69.650	3¾	6¼	3.16	750	1975	O. K.
	4	5.700	22.750	71.330	4½	6¾	3.17	1025	3350	O. K.
	5	5.450	21.800	72.490	4½	6¾	3.13	600	2230	O. K.
	6	6.960	22.850	70.110	....	....	3.12	813	2750	O. K.
F	1	5.950	20.720	73.250	3½	6½	3.12	500	2625	O. K.
	2	6.600	21.120	72.050	....	....	3.13	475	2625	O. K.
	3	6.270	21.270	72.250	3¼	7	3.12	625	2775	O. K.
	4	7.110	24.250	68.550	....	....	3.14	500	2500	O. K.
	5	6.500	22.400	71.100	4	5½	3.12	500	2310	O. K.
	6	7.000	17.020	75.700	3	4½	3.13	525	2190	O. K.
G	1	8.370	20.120	71.470	2¾	5¼	3.12	750	2125	O. K.
	2	8.300	19.600	71.800	....	....	....	875	2500	O. K.
	3	8.100	19.550	72.000	4	6½	3.09	875	2500	O. K.
	4	5.720	17.750	76.380	....	....	....	875	2500	O. K.
	5	7.200	19.030	73.600	3½	6	3.10	813	2500	O. K.
	6	6.450	18.620	74.800	....	....	....	875	2625	O. K.
H	1	8.400	19.750	71.600	....	....	3.14	688	2500	O. K.
	2	7.450	19.800	72.650	2½	4	3.13	750	2750	O. K.
	3	8.100	19.870	72.000	....	....	3.14	625	2625	O. K.
	4	7.900	20.880	71.000	3	5¼	3.12	688	2250	O. K.
	5	....	....	....	....	....	3.13	750	2250	O. K.
	6	....	....	....	3	5	3.13	625	2370	O. K.
I	1	6.950	18.900	73.900	3	5	3.09	625	3000	O. K.
	2	6.490	18.300	74.700	....	....	3.13	875	3500	O. K.
	3	....	....	....	3½	5	3.14	625	2625	O. K.
J	1	6.500	20.330	72.970	3	5¼	3.09	375	2065	O. K.
	2	6.420	18.850	74.700	....	....	3.11	438	2813	....
	3	5.900	17.500	76.500	3	5	3.12	438	2875	O. K.
	4	6.070	18.050	75.800	....	....	3.12	438	2250	....
	5	5.550	17.700	76.650	4	7¼	3.06	563	3130	O. K.
	6	5.320	17.170	77.460	....	....	....	313	1880	....
K	1	4.650	16.550	78.720	2¾	5	3.09	1125	2870	O. K.
	2	....	....	....	....	....	3.12	1250	2440	....
	3	6.490	19.050	74.490	3	5¼	....	938	2500	O. K.
	4	....	....	....	....	....	3.10	438	2630	O. K.
	5	5.850	18.520	75.430	3½	6¾	3.11	375	2500	O. K.
	6	....	....	....	....	....	3.11	438	3190	O. K.

TABLE I.—Tests of Cement and 1 : 2 Mortar. (*Continued*)

Car Number.		Percentage of Cement Retained on Sieve. 100 Mesh.	Percentage of Cement Retained on Sieve. 200 Mesh.	Percentage of Cement Through 200 Mesh.	Setting. Initial. Hrs.	Setting. Final. Hrs.	Specific Gravity	Crushing Strength of 4-in. cubes, in lb. per sq. in. 24 hrs.	Crushing Strength of 4-in. cubes, in lb. per sq. in. 7 days.	Boiling 5 hours.
L	1	8.270	29.520	72.120	2½	4	3.14	1188	2250	O. K.
	2	......	......	......	......	......	......	1188	2500	O. K.
	3	8.200	19.220	72.200	2¾	4¼	3.12	1125	2630	O. K.
	4	......	......	......	......	......	......	400	2000	O. K.
	5	7.950	18.710	73.300	2¾	4½	3.12	438	2500	O. K.
	6	......	......	......	......	......	......	475	2625	O. K.
M	1	8.110	19.040	72.750	3	5	3.12	875	2625	O. K.
	2	8.000	18.660	73.180	3½	5	3.12	1125	2375	O. K.
	3	......	......	......	......	......	......	1125	2375	O. K.
	4	......	......	......	......	......	......	1125	2625	O. K.
	5	7.000	19.520	73.250	3½	5	......	1250	3250	O. K.
	6	......	......	......	......	......	......	938	2500	O. K.
N	1	7.600	19.010	73.260	4½	6¼	3.08	538	2250	O. K.
	2	......	......	......	......	......	......	563	2375	O. K.
	3	7.680	18.520	73.570	4¼	6	3.13	563	2600	O. K.
	4	......	......	......	......	......	......	563	2875	O. K.
	5	7.499	18.400	73.880	4½	......	3.12	526	2500	O. K.
	6	......	......	......	......	......	......	438	3125	O. K.
O	1	7.090	18.950	73.900	3¾	5¾	3.13	375	2437	O. K.
	2	......	......	......	......	......	......	500	2750	O. K.
	3	7.200	19.570	73.150	4	5¾	3.12	438	2500	O. K.
	4	......	......	......	......	......	......	438	2500	O. K.
	5	7.330	20.300	72.190	3	4¾	3.13	438	2875	O. K.
	6	......	......	......	......	......	......	500	2625	O. K.
P	1	7.610	20.620	71.560	2¾	......	3.14	438	2000	O. K.
	2	7.730	15.370	75.700	......	......	3.11	438	1875	O. K.
	3	7.850	20.410	71.590	......	......	3.13	438	2065	O. K.
Q	1	6.400	20.900	72.620	3¼	5½	3.13	500	2375	O. K.
	2	7.000	21.210	71.780	4	5¾	3.12	513	2250	O. K.
	3	7.720	18.050	73.650	4½	5½	3.09	576	2275	O. K.
R	1	7.500	20.680	71.620	4¼	5¼	3.08	625	2500	O. K.
	2	7.100	16.750	76.030	3¼	5	3.07	638	2625	O. K.
	3	7.250	20.750	71.850	3¼	5	3.09	500	2188	O. K.
S	1	6.050	20.300	73.440	4¼	6	3.15	188	2250	O. K.
	2	4.770	20.400	74.700	4¼	6	3.12	188	2250	O. K.
	3	3.850	14.060	82.030	4¾	6¼	3.12	250	2250	O. K.
T	1	4.480	17.350	78.150	5	6¾	3.12	250	2688	O. K.
	2	5.910	18.780	75.250	......	......	3.12	188	2250	O. K.
	3	6.710	23.730	69.500	7¼	8	3.12	188	2313	O. K.
U	1	6.940	20.030	72.550	5½	8	3.11	250	2188	O. K.
	2	7.070	19.300	73.400	6¼	8½	3.11	250	2250	O. K.
	3	7.320	19.500	73.170	5½	8	3.13	250	2250	O. K.
V	1	7.300	17.700	75.000	5½	8½	3.12	188	1875	O. K.
	2	7.610	18.860	73.500	7	8½	3.13	188	1750	O. K.
	3	7.560	21.650	70.550	5¾	7	3.13	188	1875	O. K.
W	1	6.200	18.500	75.150	5¾	7	3.11	313	2000	O. K.
	2	6.350	18.000	75.470	5¾	7	3.13	313	1937	O. K.
	3	6.530	20.940	72.510	8½	9	3.14	250	1750	O. K.
X	1	6.490	19.040	74.320	8¾	9½	3.12	313	1937	O. K.
	2	6.440	19.530	74.020	5¾	6¾	3.13	250	1750	O. K.
	3	7.030	18.910	73.850	5¼	6½	3.14	288	2375	O. K.
Y	1	6.330	18.650	74.970	6¼	8	3.14	275	2125	O. K.
	2	6.430	18.780	74.740	6	8	3.12	275	2312	O. K.
	3	6.920	18.790	74.200	6	8¼	3.13	313	2500	O. K.

TABLE I.—Tests of Cement and 1 : 2 Mortar. (*Continued*)

Car Number.		Percentage of Cement Retained on Sieve. 100 Mesh.	Percentage of Cement Retained on Sieve. 200 Mesh.	Percentage of Cement Through 200 Mesh.	Setting. Initial. Hrs.	Setting. Final. Hrs.	Specific Gravity	Crushing Strength of 4-in. cubes, in lb. per sq. in. 24 hrs.	Crushing Strength of 4-in. cubes, in lb. per sq. in. 7 days.	Boiling 5 hours.
Z	1	6.630	18.450	74.900	6¼	8½	3.13	313	2375	O. K.
	2	7.030	19.000	73.960	5	6¼	....	438	2063	O. K.
	3	7.240	19.280	73.330	5¼	6¼	....	250	2125	O. K.
AA	1	7.100	21.280	71.470	7½	8½	3.12	138	1250	O. K.
	2	6.980	23.500	69.100	8½	9	3.11	200	1625	O. K.
	3	7.350	19.430	73.140	5	6	3.12	200	1688	O. K.
BB	1	6.160	18.110	74.670	8	8½	3.14	125	1250	O. K.
	2	7.000	19.730	73.250	8	8½	3.13	188	1500	O. K.
	3	6.440	19.270	74.230	8¾	9½	3.13	200	1625	O. K.
CC	1	6.600	21.330	71.980	9	10	3.09	125	1437	O. K.
	2	6.350	18.650	74.800	8½	....	3.12	125	1250	O. K.
	3	6.380	22.140	71.450	7¾	8½	3.12	125	875	O. K.
DD	1	4.950	17.360	77.650	6	7	3.11	125	1813	O. K.
	2	5.110	18.550	76.220	6	7	3.14	150	1188	O. K.
	3	5.000	16.960	77.960	7½	....	3.11	125	1000	O. K
EE	1	5.050	18.150	76.700	8½	....	3.12	125	1937	O. K.
	2	4.460	17.670	77.810	6	....	3.12	125	1250	O. K.
	3	4.660	20.350	74.980	6¼	....	3.07	138	1400	O. K.
FF	1	4.440	17.450	78.070	4	....	3.11	150	1400	O. K.
	2	4.730	19.480	75.770	3¾	....	3.09	138	1437	O. K.
	3	5.000	19.050	75.920	6	....	3.11	138	1313	O. K.
GG	1	5.000	19.500	74.850	7½	....	3.11	150	1437	O. K.
	2	5.250	20.650	73.700	3¼	....	3.09	150	1625	O. K.
	3	5.430	22.250	72.050	3¼	....	3.12	163	1563	O. K.
HH	1	4.960	21.750	73.030	2¼	3¼	3.14	150	1437	O. K.
	2	4.850	19.050	76.000	2¼	3¼	3.07	150	1500	O. K.
	3	4.950	19.020	75.900	3¾	4½	3.12	138	1125	O. K.
II	1	5.140	19.080	75.650	1½	2½	3.12	125	1188	O. K.
	2	5.010	20.520	74.380	5¾	....	3.11	88	1063	O. K.
	3	5.800	21.310	72.830	7	....	3.14	125	1375	O. K.
JJ	1	7.420	21.920	70.420	7½	....	3.12	125	1437	O. K.
	2	....	....	....	5¼	....	....	150	1538	O. K.
	3	7.640	23.660	68.550	5½	....	3.08	150	1563	O. K.
KK	1	10.230	23.930	65.500	6	....	3.12	188	1375	O. K.
	2	10.290	23.500	66.000	6¾	....	3.15	150	1313	O. K.
	3	9.500	24.800	65.500	6¾	....	3.15	138	1275	O. K.
LL	1	7.030	22.250	70.600	6	8	3.12	188	1563	O. K.
	2	8.200	23.900	67.900	6	8	3.12	150	1500	O. K.
	3	6.300	20.370	73.230	6	7½	3.11	138	1625	O. K.
MM	1	9.350	22.500	68.000	6	7½	3.15	125	1313	O. K.
	2	10.320	23.930	65.580	6	....	....	138	1313	O. K.
	3	10.520	23.610	65.700	....	....	3.14	188	1375	O. K.
NN	1	6.850	27.170	65.740	7	....	3.062	188	1125	....
	2	6.850	25.900	67.000	7	....	3.107	163	1437	....
	3	7.120	23.230	69.480	7	....	3.137	150	1375	....
OO	1	6.750	23.450	69.650	7	....	3.137	125	1250	....
	2	7.660	24.260	67.800	8½	....	3.122	100	1313	....
	3	6.980	22.930	70.000	8½	....	3.137	125	1250	....
PP	1	7.370	22.830	69.600	9	....	....	75	1125	....
	2	7.020	21.450	71.300	9	....	....	38	1250	....
	3	7.050	21.800	71.040	7	....	....	38	1375	....

TABLE I.—Tests of Cement and 1 : 2 Mortar. (*Continued.*)

Car Number.		Percentage of Cement			Setting.		Specific Gravity	Crushing Strength of 4-in. cubes, in lb. per sq. in.		Boiling 5 hours.
		Retained on Sieve.		Through 200 Mesh.	Initial. Hrs.	Final. Hrs.		24 hrs.	7 days.	
		100 Mesh.	200 Mesh.							
QQ	1	6.700	22.570	70.450	7	....	....	125	1625	....
	2	5.670	21.180	72.960	8	....	....	125	1688	....
	3	6.860	21.900	70.900	8	....	....	150	1625	....
RR	1	5.000	17.750	76.800	8	....	....	150	1750	....
	2	7.100	24.100	68.680	8	....	....	113	1875	....
	3	4.840	18.750	76.280	9	....	....	125	1875	....
SS	1	7.800	23.320	68.630	8	9	....	163	1375	....
	2	7.660	22.450	69.730	9	....	3.122	163	1375	....
	3	7.300	22.150	70.440	9	....	3.107	150	2000	....
....		....	....	....	....	....	....	150	1875	....

and the strength which can be safely counted on in construction is rather above than below the average of the tests made.

REQUIREMENTS FOR MATERIALS AND CONCRETE.

The values given in Tables III and IV are not necessarily true of all concrete, and perhaps it would not be amiss to describe in some detail how this concrete was made.

In order that good concrete may be produced the first requirement is that the specifications be correct, full, exact and clear. This enables the inspector to do his work on a definite plan without the annoyance of possible objections or interference by the contractor's superintendent, who is sometimes more interested in the quantity than the quality of the product. With this object in view the specifications were made for this work. Following are the items applying to the materials and manufacture of concrete:

Concrete will be composed of one (1) part cement, two (2) parts sand and four (4) parts stone, with sufficient water to make a wet mixture. The cement, sand and stone will be introduced into a batch mixer of approved design and the mixer given 3 turns before the water is added. After the water is added the mixer will be revolved at least 12 times and to the satisfaction of the engineer before the batch is dumped. Concrete must be placed immediately after mixing and well spaded to insure a dense concrete.

Cement, where used in these specifications, will mean Portland cement of such quality as to meet the standard specifications of the American Society for Testing Materials, and stand the tests prescribed in the rules and regulations of the Bureau of Buildings, Borough of Brooklyn, City of New York.

TABLE II.—Tests of Cement and 1 : 2 Mortar.

Car Number.		Percentage of Cement Retained on Sieve. 100 Mesh.	Percentage of Cement Retained on Sieve. 200 Mesh.	Percentage of Cement Through 200 Mesh.	Setting. Initial. Hrs.	Setting. Final. Hrs.	Specific Gravity	Crushing Strength of 4-in. cubes, in lb. per sq. in. 24 hrs.	Crushing Strength of 4-in. cubes, in lb. per sq. in. 7 days.	Boiling 5 hours.
A	1	5.93	18.97	74.90	2¼	3	3.152	450	1938	O. K.
	2	6.	19.15	74.70	2¼	3	3.122	563	2375	O. K.
	3	6.28	20.20	73.50	2¼	2¾	3.137	469	2125	O. K.
B	1	6.36	19.26	74.26	2¼	3	3.152	750	2313	O. K.
	2	5.75	18.41	75.75	2	2½	3.152	844	2188	O. K.
	3	6.	18.71	75.22	1¾	2¼	3.137	875	2563	O. K.
C	1	6.7	19.85	73.25	2¼	2¾	3.122	688	2125	O. K.
	2	7.26	19.90	72.70	1¾	2¼	3.137	513	1750	O. K.
	3	6.75	19.90	73.22	2	2¾	3.137	594	1813	O. K.
D	1	6.14	20.21	73.43	2	2¾	3.137	406	2000	O. K.
	2	5.94	19.82	74.01	2	3	3.168	406	1938	O. K.
	3	6.09	15.65	77.94	2	3¼	3.152	406	2063	O. K.
E	1	6.32	20 02	73.51	2	2¾	3.152	313	1875	O. K.
	2	6.5	18.20	75.21	2	3	3.122	469	1938	O. K.
	3	6.17	19.21	74.48	2	2¾	3.122	388	2000	O. K.
F	1	5.38	20.02	74.39	3¼	4¼	3.152	475	1875	O. K.
	2	6.67	16.98	76.09	3¼	4¼	3.152	388	1688	O. K.
	3	6.41	19.36	74.06	3¼	4	3.168	438	1938	O. K.
G	1	5.07	19.83	74.86	3¼	4	3.152	375	2125	O. K.
	2	6.32	20.13	73.38	3	3¾	3.152	525	2813	O. K.
	3	6.39	20.38	73.08	3	4	3.137	531	1813	O. K.
H	1	6.61	18.84	74.29	2½	3½	3.137	531	2188	O. K.
	2	5.13	19.06	75.58	2½	3¼	3.152	563	2188	O. K.
	3	5.92	19.41	74.43	2½	3½	3.152	531	2000	O. K
I	1	6.17	18.92	74.68	3	3¾	3.168	531	2125	O. K.
	2	5.06	19.09	75.53	3½	4½	3.152	525	2125	O. K.
	3	6.18	19.32	74.29	3	3¾	3.168	513	2188	O. K.
J	1	6.13	18.84	74.81	3¼	4¼	3.137	500	2125	O. K.
	2	7.73	18.96	73.08	3	4½	3.168	625	2250	O. K.
	3	7.20	17.64	74.93	2¾	4	3.137	563	2250	O. K.
K	1	6.17	17.86	75.61	3	4¼	3.152	438	2125	O. K.
	2	5.81	19.07	74.89	2½	4	3.152	388	2188	O. K.
	3	6.13	19.20	74.38	2¾	4	3.168	538	2000	O. K.
L	1	5.86	18.42	75.41	2¾	4	3.137	450	2125	O. K.
	2	5.12	18.63	75.93	3½	4½	3.168	531	2063	O. K.
	3	6.09	19.13	74.59	2¾	4¼	3.168	325	2000	O. K.
M	1	5.16	19.13	75.38	3¼	4	3.168	613	2313	O. K.
	2	6.03	18.79	74.92	3¼	4	3.152	688	2375	O. K.
	3	5.82	19.03	74.97	3¼	4¼	3.152	550	2063	O. K.
N	1	5.20	19.41	75.13	3½	4½	3.152	525	2125	O. K.
	2	5.43	19.21	75.19	3½	4½	3.137	600	2438	O. K.
	3	6.12	19.02	74.70	3¼	4¼	3.152	538	2188	O. K.
O	1	6.12	18.80	74.91	3¾	4¾	3.168	563	2125	O. K.
	2	5.87	19.21	74.79	3½	4¾	3.168	550	1875	O. K.
	3	6.02	18.84	75.01	3½	4¾	3.152	775	2313	O. K.
P	1	5.82	19.63	74.38	3¾	5	3.168	594	2250	O. K.
	2	6.45	19.47	73.43	4	5	3.168	525	1938	O. K.
	3	8.24	17.14	74.14	4	5	3.137	525	1938	O. K.
Q	1	7.42	17.39	75.02	3½	4¼	3.152	313	2125	O. K.
	2	9.76	19.31	70.80	4	4¾	3.152	250	1875	O. K.
	3	7.40	18.39	74.03	3¼	4	3.137	450	2250	O. K.

TABLE II.—Tests of Cement and 1 : 2 Mortar. (*Continued*)

Car Number.		Percentage of Cement			Setting.		Specific Gravity	Crushing Strength of 4-in. cubes, in lb. per sq. in.		Boiling 5 hours.
		Retained on Sieve.		Through 200 Mesh.	Initial. Hrs.	Final. Hrs.		24 hrs.	7 days.	
		100 Mesh.	200 Mesh.							
R	1	6.47	18.33	75.02	4	4¾	3.137	300	2250	O. K.
	2	9.50	16.24	74.08	3½	4¾	3.168	300	1938	O. K.
	3	8.96	16.88	74.05	4¼	5	3.152	219	1875	O. K.
S	1	8.66	17.20	74.02	3¾	4½	3.168	300	1625	O. K.
	2	9.34	16.70	73.81	3½	4¾	3.168	225	1750	O. K.
	3	8.12	16.91	74.83	4½	5½	3.152	219	2000	O. K.
T	1	6.79	18.62	74.41	5¼	6¾	3.137	125	1875	O. K.
	2	7.04	17.47	75.30	5¼	6½	3.122	100	1625	O. K.
	3	8.48	16.60	74.81	5¼	6¼	3.168	113	1875	O. K.
U	1	8.43	17.05	74.37	6	7¾	3.152	113	1938	O. K.
	2	5.87	18.92	74.97	5¾	8	3.152	106	2188	O. K.
	3	7.70	17.84	74.29	5¾	7½	3.168	88	1688	O. K.
V	1	6.88	17.92	74.94	4½	6	3.122	475	1963	O. K.
	2	7.13	18.22	74.47	3¾	5	3.152	438	1875	O. K.
	3	6.84	18.39	74.51	3¾	5	3.152	438	2000	O. K.
W	1	7.31	18.06	74.48	3¾	5	3.122	419	2125	O. K.
	2	8.48	14.70	76.59	3	4¾	3.152	425	2313	O. K.
	3	8.82	15.93	74.97	3	4¾	3.152	288	2063	O. K.
X	1	9.03	16.17	74.62	4½	6	3.137	213	1938	O. K.
	2	7.14	17.09	75.49	4¾	6¼	3.168	181	1875	O. K.
	3	8.23	16.18	75.44	4½	6	3.168	150	1688	O. K.
Y	1	7.13	17.95	74.79	4¾	6	3.152	169	1750	O. K.
	2	8.08	16.59	75.06	4½	6	3.137	325	2188	O. K.
	3	8.32	18.17	73.29	4¾	6¼	3.122	294	2000	O. K.
Z	1	7.93	18.31	73.49	4½	6¼	3.137	263	1625	O. K.
	2	8.12	18.22	73.43	4¾	6½	3.152	250	1437	O. K.
	3	6.92	18.05	74.88	4½	6½	3.137	288	1563	O. K.
AA	1	8.72	18.39	72.74	4¾	6¼	3.122	288	1563	O. K.
	2	7.88	18.27	73.60	4½	6½	3.122	350	1688	O. K.
	3	9.47	17.62	72.79	4¾	6¾	3.152	350	1437	O. K.
BB	1	11.40	18.70	69.78	5	6½	3.152	363	1437	O. K.
	2	9.13	18.99	71.64	5¼	6½	3.137	344	1500	O. K.
	3	8.72	18.04	73.07	5¼	6½	3.137	213	1437	O. K.
CC	1	6.93	19.09	73.84	4¼	5¾	3.137	219	1813[2]	O. K.
	2	8.31	19.01	72.43	4½	6	3.107	238	2063[2]	O. K.
	3	8.43	18.77	72.63	4¼	5¾	3.122	282	1875[2]	O. K.
DD	1	8.70	17.34	73.84	4½	5¾	3.137	338	2063[2]	O. K.
	2	7.93	18.04	73.71	4¼	5½	3.137	344	2000[2]	O. K.
	3	8.41	18.57	72.83	4½	5½	3.122	350	2375[2]	O. K.
EE	1	12.24	16.84	70.79	4¾	6	3.152	275	2063	O. K.
	2	9.17	17.32	73.24	4¾	5¾	3.152	344	2188	O. K.
	3	8.41	18.69	72.74	6¼	7	3.137	313	2250	O. K.
FF	1	9.96	17.58	72.29	6	7¼	3.122	319	2000	O. K.
	2	9.22	17.93	72.61	6½	8	3.137	244	1875	O. K.
	3	8.43	18.34	73.02	6¼	7¼	3.137	250	2000	O. K.
GG	1	10.92	18.63	70.12	6	7½	3.184	175	1563	O. K.
	2	13.30	18.95	67.50	5½	7	3.122	275	1375	O. K.
	3	11.75	18.72	69.06	5¾	7¼	3.122	100	1563	O. K.
HH	1	4.75	19.31	75.9	2	3¼	3.12	938	2500	O. K.
	2	5.28	24.87	69.72	2¼	3	3.2	1063	2000	O. K.
	3	5.15	21.62	73.12	2¼	3¼	3.152	1063	3063	O. K.

[2] Eight days.

TABLE II.—Tests of Cement and 1 : 2 Mortar. (*Continued*)

Car Number.		Percentage of Cement Retained on Sieve. 100 Mesh.	Percentage of Cement Retained on Sieve. 200 Mesh.	Percentage of Cement Through 200 Mesh.	Setting. Initial. Hrs.	Setting. Final. Hrs.	Specific Gravity	Crushing Strength of 4-in. cubes, in lb. per sq. in. 24 hrs.	Crushing Strength of 4-in. cubes, in lb. per sq. in. 7 days.	Boiling 5 hours.
II	1	5.36	22.28	72.19	2½	3	3.122	1063	2688	O. K.
	2	7.82	26.06	65.96	2½	3	3.152	938	2750	O. K.
	3	6.47	24.63	68.79	2	3¼	3.14	688	2063	O. K.
JJ	1	4.75	25.93	69.16	2½	3	3.137	500	1813	O. K.
	2	5.46	26.97	67.44	2¼	2¾	3.152	688	2000	O. K.
	3	5.83	24.66	69.39	2½	3	3.137	625	1813	O. K.
KK	1	5.00	22.72	72.16	2¼	3	3.152	688	1938	O. K.
	2	5.13	21.87	72.89	2¼	3	3.140	875	2625	O. K.
	3	5.09	22.36	72.47	2¼	3	3.137	750	2188	O. K.
LL	1	6.64	23.01	70.24	2	2¾	3.137	938	1875	O. K.
	2	6.23	23.18	70.47	2	2¾	3.137	813	2500	L. K.
	3	6.06	21.61	72.20	2	2½	3.152	938	2250	O. K.
MM	1	5.18	23.64	71.06	2	2½	3.137	813	2500	O. K.
	2	6.05	24.16	69.63	1¾	2¼	3.152	1063	3125	O. K.
	3	6.13	24.42	69.30	1¾	2¼	3.152	875	2375	O. K.
NN	1	5.97	23.19	70.71	2	2½	3.168	688	2125	O. K.
	2	6.13	22.94	70.85	1¾	2½	3.152	625	2750	O. K.
	3	5.99	21.82	72.11	2	2½	3.152	1063	2625	O. K.
OO	1	7.10	21.78	71.01	2	2½	3.152	1000	2313	O. K.
	2	5.94	22.31	71.61	2	2½	3.152	1063	2313	O. K.
	3	6.12	22.14	71.60	2	2¼	3.160	938	2375	O. K.
PP	1	5.68	25.04	69.15	2	3¼	3.137	938	3063	O. K.
	2	6.32	22.76	70.79	1¾	3¼	3.152	938	2875	O. K.
	3	6.29	24.07	69.51	2	3	3.137	1063	2750	O. K.
QQ	1	6.13	22.77	71.01	2	3	3.184	1125	2313	O. K.
	2	6.07	22.59	71.20	2	2½	3.168	1000	3250	O. K.
	3	5.84	21.57	72.28	1¾	2¼	3.168	938	2563	O. K.
RR	1	7.02	23.62	69.20	1½	2	3.184	625	2500	O. K.
	2	6.87	20.93	72.08	1½	2	3.168	500	1875	O. K.
	3	6.92	22.17	70.80	1½	2	3.168	1063	2625	O. K.
SS	1	4.53	22.05	73.05	2¼	3¼	3.137	1000	2125	O. K.
	2	4.60	20.66	74.45	2½	3½	3.152	1063	2938	O. K.
	3	4.87	21.33	73.61	2¼	3	3.152	875	2438	O. K.
TT	1	7.03	23.65	69.20	2	2¾	3.137	875	2250	O. K.
	2	6.61	23.115	69.80	2½	3¼	3.152	938	1625	O. K.
	3	6.72	22.95	70.13	2½	3¼	3.152	875	1750	O. K.
UU	1	5.87	22.06	71.91	1½	2	3.137	875	1563	O. K.
	2	5.69	23.12	71.03	1	1¼	3.137	750	1750	O. K.
	3	6.12	22.36	71.40	1¼	2	3.137	625	2063	O. K.
VV	1	6.35	24.13	68.70	2½	3	3.137	813	1875	O. K.
	2	5.86	23.70	70.21	2¾	3½	3.137	625	1563	O. K.
	3	5.92	23.15	70.49	2½	3¼	3.152	813	1875	O. K.
WW	1	5.88	19.26	74.45	2¼	3¼	3.137	750	2375	O. K.
	2	7.44	23.01	69.53	2¼	3¾	3.137	1000	2375	O. K.
	3	6.78	23.55	69.41	2¾	4	3.152	1000	2188	O. K.
XX	1	5.97	23.82	70.03	3	4¼	3.137	875	2000	O. K.
	2	6.31	23.77	69.61	2½	4	3.137	688	1563	O. K.
	3	6.41	21.95	71.39	2¾	4	3.137	625	1875	O. K.
YY	1	6.74	20.65	72.40	2½	3¼	3.122	625	1500	O. K.
	2	6.37	22.06	71.29	2½	3¼	3.137	875	2438	O. K.
	3	7.01	20.85	71.98	2½	3¼	3.137	750	2000	O. K.

TABLE II.—Tests of Cement and 1 : 2 Mortar. (*Continued*)

Car Number.		Percentage of Cement Retained on Sieve. 100 Mesh.	Percentage of Cement Retained on Sieve. 200 Mesh.	Through 200 Mesh.	Setting. Initial. Hrs.	Setting. Final. Hrs.	Specific Gravity	Crushing Strength of 4-in. cubes, in lb. per sq. in. 24 hrs.	Crushing Strength of 4-in. cubes, in lb. per sq. in. 7 days.	Boiling 5 hours.
ZZ	1	5.91	18.22	75.65	2¼	3	3.168	938	2438	O. K.
	2	6.69	15.42	77.75	2¼	3	3.155	688	2250	O. K.
	3	6.21	15.31	78.14	1¾	2¾	3.122	625	2375	O. K.
AB	1	8.06	17.93	73.60	2	3	3.152	563	1688	O. K.
	2	5.52	17.83	75.33	2¼	2¾	3.152	375	2000	O. K.
	3	7.02	20.05	72.70	2	2¾	3.152	375	1750	O. K.
AC	1	9.20	20	70.66	2	3¼	3.152	1250	2375	O. K.
	2	7.21	19.03	73.07	2	3¼	3.152	1000	2250	O. K.
	3	6.92	17.95	75	2	3	3.137	1250	2875	O. K.
AD	1	8.35	19.35	72.12	3½	4	3.137	1125	2125	O. K.
	2	7	19.80	73.21	3¼	3¾	3.137	1188	2063	O. K.
	3	7.47	21.02	71.51	3½	4	3.137	1063	2438	O. K.
AE	1	6.84	19.35	73.60	3¼	3¾	3.137	813[1]	2375[1]	O. K.
	2	7	20	72.89	3	3½	3.137	938	2125	O. K.
	3	6.30	19.27	74.35	3	3½	3.137	938	2250	O. K.
AF	1	6.44	18.45	75	2¾	3	3.152	813	1750	O. K.
	2	6.55	18.56	74.83	2¾	3	3.152	500	2188	O. K.
	3	6.9	18.8	74.07	3	3½	3.137	438	2313	O. K.
AG	1	6.7	18.71	74.55	3¼	4	3.137	688	1750	O. K.
	2	6.12	19.06	74.61	3	4	3.137	625	1688	O. K.
	3	7.13	19.28	73.40	3	4	3.152	781	1750	O. K.
AH	1	6.23	19.66	74.05	2¾	4¼	3.152	656	2250	O. K.
	2	5.47	19.80	74.58	2¾	4	3.137	875	2250	O. K.
	3	7.11	18.80	73.87	3	4¼	3.137	781	2438	O. K.
AI	1	7.92	20.08	71.80	3	4¼	3.137	656	2125	O. K.
	2	8.46	21.19	70	2¾	4¼	3.137	719	2063	O. K.
	3	8.17	20.58	70.71	3½	5	3.137	563	2250	O. K.
AJ	1	7	20.47	72.32	3	4	3.137	531	2000	O. K.
	2	6.77	21.21	71.80	3¼	4¼	3.137	563	2063	O. K.
	3	5.76	19.58	74.53	2½	3	3.137	594	2188	O. K.
AK	1	6.09	19.66	74	2½	3	3.152	500	1938	O. K.
	2	6.16	19.98	73.74	2¼	3	3.137	375	1625	O. K.
	3	6.04	19.82	74.09	2¼	3	3.152	594	2250	O. K.
AL	1	5	19.45	75.35	2	3	3.168	531	2188	O. K.
	2	5	20.21	74.70	2¼	3¼	3.137	500	2313	O. K.
	3	5.25	20.19	74.24	2	3½	3.152	406	1688	O. K.
AM	1	6	19.97	73.63	1¾	2½	3.122	638	2188	O. K.
	2	6.82	18.20	72	2	2½	3.137	663	2188	O. K.
	3	7.37	18.42	74.03	2	2¾	3.137	513	1813	O. K.
AN	1	6.82	20	72.80	1¾	2½	3.152	388	1750	O. K.
	2	6.53	19.75	73.45	1¾	2½	3.152	531	2000	O. K.
	3	6.41	19.90	73.45	2	3¼	3.137	375	1813	O. K.
AO	1	5.95	19.40	74.50	1¾	3	3.122	475	2750	O. K.
	2	6.25	19.05	74.60	2	3	3.137	469	2250	O. K.
	3	6.42	20.11	73.35	1¾	2¾	3.137	481	1813	O. K.

[1] Sand contained 56 per cent loam.

TABLE III.—CRUSHING STRENGTH OF 4-IN. CONCRETE CUBES.

Date made.	Strength in lb. per sq. in. 24 hr.	7 dy.	28 dy.	Str'gth in lb. per sq. in.	Age in days.	Date made.	Strength in lb. per sq. in. 24 hr.	7 dy.	28 dy.	Str'gth in lb. per sq. in.	Age in days.
8-18-10	375	1250	2425	3688	336	11-17-10	250	2125[4]	3500[6]	4688	240
8-18-10	375	....	2475	3566	336	11-18-10	188	1750	2750[7]	5000	241
8-19-10	500	1375	2500	4688	335	11-19-10	313	1250	4250	5250	252
8-19-10	375	1900[1]	2188	3750	335	11-21-10	250	1500	3375	4375	238
8-20-10	500	1125	2875	4250	334	11-29-10	275	1875	3500	4750	230
8-22-10	500	1250	2225	4500	331	11-30-10	188	1250	3725	4875	226
8-25-10	625	2000	2875	4375	329	12- 1-10	200	1875	3125	4375	224
8-25-10	500	2000	3938	5625	328	12- 1-10	200	1875	4938	6750	239
8-26-10	375	1350	2875	4375	327	12- 2-10	200	1875	3188	4625	223
8-26-10	375	1750	2875	4250	322	12- 5-10	263	1250	2750	4875	225
8-30-10	375	875	3000	4375	323	12- 8-10	188	1500	3188	5000	221
8-31-10	750	2275	3500	4625	321	12-15-10	200	2250	3438	4750	214
9- 6-10	875	1975	3500	5000	316	12-19-10	200	2375[5]	4438	....	.....
9- 7-10	438	1688	2500	3875	316	12-20-10	350	1900	2938	4125	212
9- 8-10	563	1500	3125	4375	313	12-27-10	250	1625	3250	4875	205
9-13-10	563	1375	3125	4813	308	12-28-10	313	2375	4625[8]	6125	203
9-14-10	500	1875	3375	4625	307	12-28-10	250	1875	4188[8]	4688	201
9-14-10	438	1625	3250	5063	307	12-29-10	438	3025	5563	6125	200
9-15-10	750	1875	4375	6250	307	12-30-10	275	2750	5500	5500	200
9-15-10	625	2625	3813	4813	306	1- 9-11	188	1600	3063	4625	189
9-16-10	563	2250	3750	5563	306	1-11-11	150	1400	2938	3125	190
9-16-10	600	2188	3750	5000	305	1-12-11	400	3150	4000	7500	197
9-17-10	375	1500	3125	5250	304	1-12-11	250	1688	3750	5000	197
9-19-10	500	1938	2500	5250	303	1-14-11	250	1625	3250	4875	195
9-20-10	688	2250	3500	4500	302	1-19-11	225	1438	3125	4875	190
9-20-10	813	2250	3938	4500	301	1-19-11	313	1688	3125	4500	190
9-21-10	725	1875	3500	4938	300	1-21-11	150	1313	2625	3625	188
9-21-10	563	2125	3750	4750	301	1-23-11	125	1250	2625	....	.....
9-22-10	563	2500	3500	5875	294	1-23-11	150	1313	2500	4250	186
9-22-10	500	1875	2938	4375	299	1-24-11	338	1625	2625	3750	185
9-23-10	750	2250	2688	5375	298	1-26-11	262	1250	2375	4125	168
9-23-10	438	2000	3125	....	.....	1-26-11	438[9]	1250	2250	3500	183
9-24-10	500	2250	3125	4688	299	3- 8-11	188	1625	3250	4750	124
9-26-10	1000	1750	2500	4688	297	3- 8-11	200	1750	2875	4375	124
9-27-10	600	1875	3500	4188	294	3- 9-11	175	1938	2875	....	.....
9-27-10	875	1875	3125	4625	296	3- 9-11	225	1975	3938	4875	124
9-28-10	563	1575	2750	3750	294	3-10-11	188	1875	3750	5813	132
9-29-10	500	2063	3000	4250	292	3-10-11	263	2375	5625	5625	121
9-30-10	563	1875	2875	4125	293	3-11-11	250	2500	5000	7188	123
10- 3-10	250	1250	2000	3125	288	3-13-11	313	2188	4813	6500	121
10- 4-10	500	2125	3125	4938	289	3-14-11	250	2750	4625	6750	128
10-11-10	500	2000	2750	3500	279	3-14-11	188	2125	4188	6250	118
10-13-10	375	1875	2813	4313	277	3-18-11	188	2250	5500	6125	116
10-14-10	500	1750	2750	4250	276	3-20-11	350	2500	4750	7125	114
10-15-10	375	1875	2250	3563	273	3-20-11	288	2500	4375	5625	114
10-17-10	500	2250	2938	4813	274	3-21-11	375	3000	4750	6625	113
10-19-10	500	1250	2000	4375	268	3-21-11	275	2500	5000	5875	112
10-19-10	688	1875	2750	4750	269	3-23-11	313	2375	5000	5750	110
10-25-10	313	1250	2250	3625	261	3-30-11	250	2500	4625	6750	103
10-25-10	313	1375	2250	3125	262	3-31-11	150	2000	4875	6250	101
10-26-10	375	2063	2688	4625	266	4- 1-11	313	2563	5250	7500	102
10-28-10	350	1688	2938	4875	264	4- 3-11	250	2500	4625	6125	100
10-31-10	250	1563	2000	4375	256	4- 3-11	250	2625	5000	5125	100
10-31-10	263	1250	2750	4750	256	4- 5-11	250	2125	4625	6000	98
10-31-10	313	2125	3125	....	.....	4- 6-11	375	2563	4500	5125	96
11- 1-10	313	1750	2750	4938	260	4-13-11	375	3125	4625	6625	89
11- 2-10	500	1250	2875	5875	251	4-13-11	438	3125	4750	6125	88
11- 2-10	725	2125	2875	4063	258	4-14-11	375	2750	4625	5625	89
11- 7-10	350	2438	4500	6500	250	4-14-11	438[10]	3375[5]	5625	4750	89
11- 7-10	313	2250	4125	6063	255	4-15-11	563[11]	2250	3500	4375	34
11- 9-10	313	2250	3125	5500	246	4-18-11	375	1625	3500	4625	85
11-10-10	350	1625	3125	4750	247	4-22-11	250	1625	3375	4063	79
11-10-10	375	1500	3000	6000	246	4-25-11	375	1875	3750	4750	78
11-11-10	250	2000	3125	4125	245	4-27-11	438	2375	4375	6125	76
11-11-10	188	2000	3125	5125	250	4-28-11	750	2500	5000	6875	74
11-12-10	250	2125	2750[3]	5875	248	5- 8-11	500	2250	4125	5000	64
11-14-10	188	1375	2375	4250	243	5-12-11	625	1875	4375	5000	60
11-15-10	225	1750	3813	5875	246	5-18-11	625	2250	4375	5375	54
11-16-10	188	1250	2500	4188	241	5-20-11	875	1750	3625	4063	52
11-16-10	313	2125[4]	3125	....	.....						

[1] 14 days.
[2] 30 hours.
[3] 30 days.
[4] 12 days.
[5] 8 days.
[6] 70 days
[7] 27 days.
[8] 29 days.
[9] 43 hours.
[10] 21 hours.
[11] 53 hours.

TABLE IV.—CRUSHING STRENGTH OF 4-IN. CONCRETE CUBES.

Date made.	Strength in lb. per sq. in.			Str'gth in lb. per sq. in.	Age in days.	Date made.	Strength in lb. per sq. in.			Str'gth in lb. per sq. in.	Age in days.
	24 hr.	7 dy.	28 dy.				24 hr.	7 dy.	28 dy.		
7-15-11	1000[12]	1500	2188	3625	180	9-25-11	719	1375	2250	4500	108
7-17-11	563	1750	2875	3950	116[1]	9-25-11	650	1313	2313	4000	108
7-18-11	688	1938	3375	4875	177	9-27-11	688	1625[5]	2438	4625	106
7-18-11	750	1688	3750	4519	115[1]	9-28-11	388	1000	2000	3625	105
7-19-11	688	2000	3188	5000	176	9-28-11	344	875	2188	3625	105
7-24-11	750	1625	3250	4031	109[1]	10- 2-11	263	938	2313	3813	101
7-25-11	625	1750	3188	5063	170	10- 2-11	294	875	2063	3625	101
7-25-11	563	1500	2438	4000	170	10- 3-11	469	1188	2563	4375	100
7-26-11	563	1375	2375	4313	107[1]	10- 3-11	438	1125[8]	2063	3688	100
7-26-11	625	1625	2875	4688	169	10- 5-11	419	1000	2250	3625	98
7-31-11	625	1375[14]	2813	4269	102[1]	10- 6-11	438	1063	2813	4250	97
8- 1-11	625	1375	2688	. . . .	. . . . .	10-10-11	438	1063	2188[8]	4250	93
8- 1-11	688	1250	2563	2938	38	10-10-11	388	1188[8]	1875[7]	3563	93
8- 2-11	600	1250[2]	2438	3813	100[1]	10-12-11	375	1063	2000	3688	91
8- 2-11	625	1500[3]	2250	3875	100[1]	10-12-11	375	875	1938	3875	91
8- 3-11	688	1375	3000	4069	99[1]	10-16-11	406	1000	2063	4125	87
8- 3-11	688	1313	2438	4519	99[1]	10-17-11	375	969	2313	4313	86
8- 4-11	625	1250	2750	5125	[4]98[1]	10-17-11	450	1500[6]	2438	4188	86
8- 4-11	500	1188	2500	4169	[4]98[1]	10-19-11	325	788	2125	4063	84
8- 8-11	563	1500	2875	4288	94[1]	10-20-11	294	825	1875	3375	83
8- 9-11	813	1938[5]	3438	5625	155	10-23-11	300	788	2188	3438	80
8-10-11	563	1063	2125	3625	154	10-24-11	344	919	2063	3875	79
8-11-11	625	1063	2188	3637	91[1]	10-25-11	263	800[5]	1813	3688	78
8-11-11	500	875	1938	3875	153	10-26-11	263	675	1750	3375	77
8-14-11	563	1125	2125	3719	88[1]	10-30-11	263	888	2875	5125	73
8-14-11	625	1188	2500	3875	150	10-31-11	300	969	2563	5063	72
8-16-11	438	1125[5]	2250	3438	86[1]	11- 1-11	238	600[5]	2125	3625	71
8-18-11	563	1188	2563	4063	146	11- 2-11	113[10]	469	2063	3875	70
8-25-11	563	1313	2313	3788	77[1]	11- 2-11	100[10]	400	2188	3875	70
8-28-11	625	1125	2188	3750	136	11- 3-11	. . . .[11]	656[5]	1625	3625	69
8-28-11	563	1000	1813	3138	136	11- 4-11	250[12]	600	2438	3438	68
9- 1-11	563	1375	3188	4813	132	11- 6-11	400[12]	781	1625	3438	66
9- 1-11	375	1250	2625	4375	132	11-11-11	450[12]	663	2000	2875	61
9- 2-11	1250[6]	1188	2813	3938	131	11-13-11	94	438	1625	3250	59
9- 5-11	625	1250	2500	4625	128	11-13-11	169	588	1938	3000	59
9- 7-11	500	1125	2625	3688	126	11-14-11	250	713	1625	3000	58
9- 8-11	375	875	2063	3875	125	11-14-11	275	788	1938	3250	58
9-12-11	563	1063	2688	5000	121	11-17-11	125	875[15]	1813	3125	55
9-13-11	375	1063[2]	2563	4125	120	11-20-11	225	800	1625	2813	52
9-14-11	500	1063	2438	4375	119	11-20-11	194	888	1813	3125	52
9-16-11	938[7]	1375	2563	4500	117	11-21 11	331	1125	2000	3125	51
9-18-11	325	1000	2250	3938	115	11-22-11	100	813	2000	3000	50
9-18-11	450	1188	2250	3813	115	11-24-11	175	750	3063	4438	48
9-19-11	487	1188	2375	4250	114	11-28-11	200	1000	1938	3063	44
9-20-11	413	1250[5]	1938	3438	113	11-29-11	[11]188[12]	688	1688	2500	43
9-20-11	438	1500[5]	2250	4063	113	11-29-11	[11]156[12]	875[13]	1813	2188	43
9-21-11	438	1500	2563	4250	112	12- 1-11	219	719	1563	2625	41
9-22-11	1500[6]	1688	2875	4875	111	12- 1-11	156	869	1813	3250	41

[1] Broken at Columbia University.

[2] 5 days, 7 hours.

[3] 5 days, 2 hours.

[4] Contained 5 per cent loam, extra 10 per cent cement used.

[5] 5 days.

[6] 3 days.

[7] 53 hours.

[8] 6 days.

[9] 29 days.

[10] Cube not set.

[11] Cube broke when removed from forms.

[12] 48 hours.

[13] 9 days.

[14] 4 days.

[15] 7 days.

All sand must be washed, clean, sharp, silicious and free from injurious matter.

All stone shall be clean crushed trap rock, free from dust and to contain no particle that will not pass through a $\frac{3}{4}$-in. ring.

There was little difficulty in getting satisfactory materials. Standard brands of Portland cement were used. The sand was what is known in New York as "Cow Bay," and was satisfactory with a few exceptions when, for some reason, it was impossible to get washed sand from the usual source and other sands were tried temporarily to take its place in order not to delay the work. Some of this sand was satisfactory, some was condemned outright and one scow load was accepted on condition that 10 per cent additional cement be used,—a doubtful expedient. The stone was a Hudson Palisades trap, commercial $\frac{3}{4}$-in. size and proved generally acceptable.

OPERATION OF PLANT.

The mixer, a batch mixer, was set in a pit so that the top of the receiving hopper was a little above the floor of the wharf. The sand and stone were measured approximately in wheelbarrows or buggies and the cement by the bag, reckoning a bag of cement at 0.95 cu. ft. The aggregates were all put into the hopper, the trap opened and the batch pushed into the mixer. As soon as all the aggregates were clear of the hopper the water was added, the mixer revolved 12 times and dumped, while another batch was being prepared.

At first the water was dipped with a pail from a barrel on the platform and splashed through the hopper into the mixer. This kept the sides of the hopper always wet, preventing the easy discharge of the dry aggregates and also delayed the work by using the hopper as a funnel while another batch should have been in course of preparation. These were matters primarily for the contractor's attention, but this method produced concrete of such varying degrees of consistency that the plan shown in Fig. 2 was suggested to the contractor and immediately installed. By this method the water flows constantly or nearly so from a $\frac{3}{4}$-in. pipe into the upper barrel; from this barrel a $1\frac{1}{2}$-in. pipe with quick opening lever gate valve leads to the lower barrel. From the bottom of the lower barrel a 3-in. pipe, also fitted with quick

opening valve, leads into the mixer under the hopper trap. A number of small holes were bored in the side of the lower barrel. By trial the right amount of water for a batch of concrete was determined and all the holes below the line corresponding to that amount plugged. Only the inspector at the mixer was allowed to vary this amount. After all the dry material had run into the mixer, which it did more readily since the hopper was always dry, the hopper trap was closed and the 3-in. valve opened. Only a few seconds was required to empty the barrel, when the 3-in. valve was closed, the 1½-in. valve opened and the water allowed

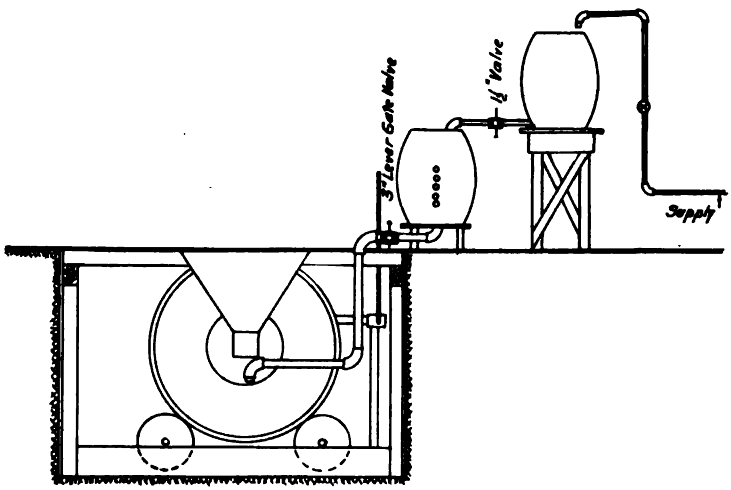

FIG. 2.—ARRANGEMENT OF WATER SUPPLY FOR MIXER.

to run until it had filled the lower barrel to the holes in the side, which served both to call the attendant's attention and to drain off surplus water. By this method a satisfactory mixture was achieved and the time of mixing reduced by at least a third.

The concrete as thus produced was of such consistency that it was wheeled to the work—at most 300 ft.—without any apparent segregation of material and when the buggy was tipped the whole mass contained slipped, rather than poured, into the forms, leaving the buggy clean. The difference between concrete that is just wet enough and concrete that is too wet is very slight

and only experience can determine what is right and constant care maintain it.

The batch was dumped from the mixer into a bucket and immediately hoisted and dumped into a hopper above the floor where it was to be used. From this hopper it was drawn off into concrete buggies and wheeled over plank runways to the work. The buggies had a capacity of 6 cu. ft., but in practice each man took about two thirds that much.

The columns were poured first, preferably 24 hours before the slab. The concrete in the columns was puddled with long wooden bars about 2 x 3 in., which were also used to hold the reinforcement in place as the concrete was being poured. Every buggyful of concrete was thoroughly puddled before the next was added.

In concreting the floors it was the general practice to fill the beams and girders first, allowing the concrete to spread on to the adjoining slab rather than from the slab into the beams and girders. The floor concrete was puddled with narrow wooden-handled iron spades. Care was taken that no concrete stood long enough to form a joint before new concrete was added. When construction joints were necessary they were made at the middle of the span and were vertical. Before new concrete was added the surface of the joint was roughened and washed with neat cement grout. The concreting gang was followed closely by the finishers who screeded the slab to the required thickness and troweled the surface to a smooth, hard finish.

While the inspection was close and strict the contractors co-operated heartily and the work was done without friction.

COMPARATIVE TESTS OF THE STRENGTH OF CONCRETE IN THE LABORATORY AND IN THE FIELD.

BY RUDOLPH J. WIG.*

It is common practice at the present time to define a desired strength of concrete by stating that it shall be made of good quality materials in certain given proportions. Assuming the use of average concrete material a 1:2:4 proportion mixture is considered as developing an ultimate compressive strength of 2000 to 3000 lb. per sq. in. in 28 days, 3000 to 3500 lb. per sq. in. in 13 weeks, etc. These values are based upon the results of many thousands of laboratory tests which undoubtedly represent the true values which can and should be obtained with good materials if properly used in the field. Much attention is given to careful selection and the testing of the materials entering into the concrete and usually in case of the failure of the concrete, the cause of failure is attributed to the cement or poor aggregate.

Several years ago an investigation was made in the Structural Materials Testing Laboratories of the United States Geological Survey at St. Louis, Mo., to determine the influence of difference in workmanship upon the strength of reinforced concrete in flexure. In connection with this investigation compression tests also were made on the plain concrete molded into 8 x 16 in. cylindrical test pieces. The work of three prominent contractors of the city of St. Louis was compared and the results of the tests are of much interest.

The materials, cement, sand and aggregate, were carefully weighed and proportioned under the supervision of the laboratory force which ensured each contractor receiving exactly the same amount of material so that the variation in the results of the tests of the concrete prepared by the contractors was due entirely to a difference in the method of mixing and handling the materials.

In the beam tests for hand-mixed concrete in the proportion

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1 part cement to 3 parts sand to 6 parts stone when 4 weeks old, there is a difference of over 200 per cent in the maximum load carried by beams made by different contractors with the same materials. The total maximum load varied from 4850 to 16,310 lb. For the machine-mixed concrete the variation is about 90 per cent ranging from 8160 to 14,500 lb. total maximum load. The quality of the concrete can perhaps be better observed from the compression tests on the cylinders which were molded from the same mixtures as the beams. The placing of the material in the cylinder molds, however, was done by the laboratory force so that the difference in the results of these tests may be entirely attributed to the difference in the method and the thoroughness of the mixing and the difference in the consistency of the concrete as mixed, as each contractor was instructed to prepare a concrete such as he used in practice and to exercise no more care in its preparation than he would under working conditions.

This privilege of varying the quantity of water perhaps had the greatest effect upon the strength although it was also materially influenced by the method of mixing as is shown in the results where two of the contractors used the same percentage of water in mixing.

A summary of the results is as follows:

1:3:6 GRAVEL CONCRETE, MACHINE MIXED. (Average of 3 test pieces.)
Compressive Strength in lb. per sq. in.

	4 weeks.			13 weeks.		
	Max.	Min.	Ave.	Max.	Min.	Ave.
Company A.........	998	650	844	1470	1180	1363
" B.........	577	513	549	1020	878	944
" C.........	725	623	679	1243	956	1138
Laboratory..........	1074	961	1032	1653	1540	1611

There is an approximate difference of 100 per cent in individual values which range from 513 to 1074 lb. per sq. in. at 4 weeks and from 878 to 1653 lb. per sq. in. at 13 weeks.

1:3:6 GRAVEL CONCRETE, HAND MIXED. (Average of 6 test pieces.)
Compressive Strength in lb. per sq. in.

	4 weeks.			13 weeks.		
	Max.	Min.	Ave.	Max.	Min.	Ave.
Company A.........	988	510	746	1030	561	807
" B.........	672	409	518	590	522	568
" C.........	640	321	475	881	640	764

There is an approximate difference of 200 per cent in individual values ranging from 321 to 988 lb. per sq. in. at 4 weeks and an approximate difference of 100 per cent at 13 weeks, the values ranging from 522 to 1030 lb. per sq. in.

1:3:6 LIMESTONE CONCRETE, HAND MIXED. (Average of 6 test pieces.)
Compressive Strength in lb. per sq. in.

	4 weeks.			13 weeks.		
	Max.	Min.	Ave.	Max.	Min.	Ave.
Company A	447	398	422	810	668	745
" B	405	364	390	584	522	556
" C	999	507	693	1380	575	864

There is an approximate difference of 200 per cent in individual values ranging from 364 to 999 lb. per sq. in. at 4 weeks, and an approximate difference of 150 per cent at 13 weeks, the values ranging from 522 to 1380 lb. per sq. in.

1:2:4 GRAVEL CONCRETE, MACHINE MIXED. (Average of 3 test pieces.)
Compressive Strength in lb. per sq. in.

	4 weeks.			13 weeks.		
	Max.	Min.	Ave.	Max.	Min.	Ave.
Company A	1331	1211	1265	1834	1675	1740
" B	1741	1461	1630	2195	2105	2157
" C	2375	1991	2216	2290	1958	2132
Laboratory	2589	2540	2572	2698	2634	2672

(All test pieces of this series were exposed to the weather.)

There is an approximate difference of 100 per cent in individual values ranging from 1211 to 2589 lb. per sq. in. at 4 weeks and of 70 per cent at 13 weeks values ranging from 1675 to 2698 lb. per sq. in.

1:2:4 GRAVEL CONCRETE, MACHINE MIXED. (Average of 3 test pieces.)
Compressive Strength in lb. per sq. in.

	4 weeks.			13 weeks.		
Company A	1650	1160	1443	....	....	1890
" B	1942	1691	1787	2607	2294	2485
" C	1966	1615	1828	2389	2220	2308
Laboratory	2487	2089	2312	2899	2735	2809

(All test pieces of this series were cured in the laboratory.)

There is an approximate difference of 130 per cent in individual values which range from 1160 to 2487 lb. per sq. in. at 4 weeks and of 50 per cent at 13 weeks, values ranging from 1890 to 2899 lb. per sq. in.

The yield point of the concrete in the last two series varied from 400 to 1000 lb. per sq. in.; thus in some cases the yield point

of the concrete at 4 weeks was actually below the usual allowable working stress of 1:2:4 concrete which is 500 lb. per sq. in. The initial modulus of elasticity varied from 2,880,000 to 5,360,000 or approximately 100 per cent.

From these results, which it is believed are substantiated by other field tests, it would seem that more attention must be given to the mixing and handling of the concrete or it is not safe to assume an ultimate compressive strength of 2000 lb. per sq. in. for 1:2:4 concrete at the age of 4 weeks. This value is readily obtained in the laboratory and by some contractors and there is no reason why it should not be obtained by all. As a general thing specifications are not sufficiently definite upon the method of mixing and the consistency to be used and too much has been left to the discretion of the engineer or foreman in charge of the work.

DISCUSSION

Mr. Wason. MR. L. C. WASON.—I would like to ask whether the laboratory or the field tests gave the higher results and whether they were uniformly one way.

Mr. Wig. MR. R. J. WIG.—The laboratory tests were always the higher with one exception. There was one contractor who approached, in fact met, the laboratory tests in most cases, but the laboratory tests were always the higher and more uniform. There was less difference between the government test pieces or beams.

Mr. Wason. MR. WASON.—Speaking from memory, during the early work of the Boston Transit Commission in subway construction, tests pieces were taken from the actual work to the laboratory and invariably gave higher results than the laboratory test pieces.

Mr. Humphrey. PRESIDENT HUMPHREY.—The object of the test was to determine if possible the relation between work done under laboratory and under field conditions. The three companies were doing what might reasonably be called first-class concrete work. The men, however, tried to do their best as they knew they were going to the laboratory to do work under competition, although an effort was made to eliminate this feeling.

The results show that no matter how conscientious a contractor may be that unless he knows in some definite way what he is doing, judgment is of no value in determining the quality of concrete.

There is probably no phase of concrete work meriting so much careful consideration at the present time as the very feature brought out by the paper of Mr. Wig. It certainly is a subject we have been hammering ever since this Association has been in existence—better concrete. The assumption of 2000-lb. concrete without knowledge whether it is obtainable, without any attempt to determine whether it is obtained, cannot be too strongly condemned. One of the essentials in concrete work, which we cannot too strongly emphasize, is the development of field inspection and tests of concrete.

Our Committee on Concrete Materials have not reported recommendations this year, but we hope they will, because any method is better than none. The best education any contractor can have at the present time is to use some simple compression machine, no matter how inexpensive as long as it gives approximate results as far as the load is concerned, that will tell whether the materials are giving a 1000- or 2000-lb. concrete, or as was shown on the screen by Mr. Kinney, whether the strength is only a little over 300 lb. per sq. in. Mr. Humphrey.

The influence of the percentage of water on the strength of concrete is not fully appreciated. The concrete can be too wet as well as too dry. It must be thoroughly mixed and water cannot take the place of mixing. Use the minimum percentage of water that you can to get a good consistency, a sticky consistency, and mix a half a minute or a minute, if possible, in the mixer, and the result will be a stronger and better concrete. The mixing must coat the aggregate and rub it together which cannot be done by merely adding water.

It is hoped that definite recommendations on standard field methods will be made by the committee next year, not methods that can be used in the laboratory, but methods that can be used on the work. This is accomplished in Austria and other parts of Europe through the control beam. It is very simple, but a rather expensive field test for the average contractor.

MR. WIG.—The concrete was all proportioned by volume and then weighed, that is, the volume measurement was transposed into weight measurement. The sand was taken from a bin containing a certain amount of moisture. The moisture was determined before admission. The quantity of gravel and sand to be used and the weight per cubic foot had been predetermined. So these results of 1 part of cement, 2 of sand and 4 of gravel, may vary slightly, depending upon the method of determining the weight per cubic foot. The weight per cubic foot was the same with either, practically, so all variables have been eliminated except those to be compared: time, strength and workmanship. Mr. Wig.

PRESIDENT HUMPHREY.—There were two series of tests, one on a 1 : 3 : 6 concrete and another on a 1:2:4. The three companies had nothing to do with the preparation or the handling of the materials. They simply fixed the consistency, which was Mr. Humphrey.

Mr. Humphrey. variable, by requesting the quantity of water. Some tests were made by hand methods and some were made by machine mixing. The average laboratory test on 1 : 3 : 6 concrete at 4 weeks was about 1000 lb. per sq. in. The companies claimed this mixture was not a practical mixture for reinforced concrete conditions and the second series was made on 1 : 2 : 4 proportions under the same conditions by the same companies. This concrete gave over 2000 lb. at 4 weeks.

Mr. Chubb. MR. J. H. CHUBB.—I would like to ask if there was any record kept of the amount of water that the particular contractors used, the consistency. It seems to me that since the variation was only in the long time tests, the result of those tests would show that nearly all those variations were due to consistency. In a long time test the variation was about half what it was in the short time test. If they ran 60 days or 6 months it would probably be much closer.

Mr. Humphrey. PRESIDENT HUMPHREY.—The amount of water used was measured. While the contractor said how much water, we measured how much water he actually used, so that the tests when they finally appear in the bulletin form of the United States Bureau of Standards, will contain the actual amount of water used in these various tests.

It is found that at the end of a year there is less variation than at any earlier period. When it is considered, however, that concrete structures are put into service in a very short time and that the contractor wants to remove the forms some in 2 days and most in 4 days, certainly a week, it must be known just how to handle the concrete to obtain the strength desirable under such conditions. The factor of safety generally increases with age as the concrete grows harder, but the important thing to know is how to make the concrete so as to get the maximum strength at an early period.

Mr. Wig. MR. WIG.—I think we have given a little too much attention to the ultimate strength of concrete rather than to the yield point and elastic properties—not so much the elastic properties as the yield point. The yield point of concrete in these tests was at a strength of 400 lb. in some cases at four weeks. The forms are usually removed in one or two weeks. In such cases the yield point would probably not be more than 150 or 200 lb. and it would

be a question whether the building would withstand its own load, its own weight. The advantage in this variation in the consistency is in the variation of the yield point. When the concrete yields it will ultimately fail. In putting the test piece in the testing machine the load was applied so rapidly that the yield point would be little affected and carried to failure. When the load is left on a little beyond the yield point the concrete will ultimately fail. **Mr. Wig.**

Usually the yield point will run around 1500 or 2000 lb.; 1000 lb. would be very ordinary, and 1000 lb. was obtained in these tests.

MR. WM. M. KINNEY.—I think that this brings us back again to the question of aggregates. Some aggregates certainly do affect the early strength more than others, and that is something that must be studied; that is, these tests must be made in order to find out the strength of the concrete in a certain length of time. Referring to the paper by Mr. Cummings,* I believe the use of steam resulted from the fact that in 14 to 16 days he could not get sufficient strength in his concrete made from a particular aggregate to drive the pile, which necessitated the use of steam in order to cure it more quickly. A job usually can not be delayed too long for foundations, and piles are the starting of a foundation, of course. **Mr. Kinney**

* See p. 312.—ED.

THE NECESSITY FOR FIELD TESTS OF CONCRETE.*

By Fritz von Emperger.†

One of the most essential precautions taken in every well regulated manufacturing plant is in the control of raw material as to its qualities in order to properly guard against poor material. Failure to do this results in a reduction of the efficiency of the finished work, causing either accident or reconstruction, which sometimes has a far-reaching effect, especially in the case of construction work where it often leads to the financial ruin of the contractor even if there is no accident. It would seem natural that in concrete construction this precaution should not be omitted.

At the present time, however, it is considered sufficient to test the cement according to standard methods and to judge the fine and coarse aggregates by visual inspection. The inadequacy of this procedure has been demonstrated several times recently through the fact that the concrete in parts of large buildings resulted so poorly that they had to be rebuilt and in another case the structure was accepted only at the combined risk of the contractor and the owner because the only correct manner of handling the matter was beyond their financial ability, that is rebuilding. In all of these cases the raw materials, although not of first class quality, were as good as those used in hundreds of other cases where no trouble occurred. It was even proven experimentally that the materials, claimed faulty by interested parties, when mixed with first-class cement gave excellent results, but when mixed with the cement used on the particular job, the results were below the requirements. Similar results have pointed to cement coming from certain mills. However, with first-class aggregates this cement produced a concrete of sufficient strength. In one case it was found that the cement had not been seasoned sufficiently, which was immediately remedied. In this case the control beam was used and it is mentioned to illustrate the effectiveness of the check on the use of proper materials.

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* Translated from the German, by the Secretary.

† Consulting Engineer, Vienna, Austria.

The difficulty with concrete control is the long time required to obtain results. The speed with which modern concrete structures are erected makes each day of great value. Therefore, the amount of time required to send the test pieces to a laboratory should be eliminated because a quick control is possible only on the job. The method given in the latter part of this paper makes it possible to obtain results within from 8 to 10 days, which will clearly indicate all poor concrete. It is hardly necessary to state that in conctruction work it is only important to detect considerable variation of the concrete from the average quality.

In the above mentioned case the conditions were about as follows: The aggregates were first class and the mixing and other operations were properly done. The first 10-day test showed only one half compressive strength and it seemed clear that the trouble was most likely with the cement. In the meantime one story had been built with such concrete. The test indicated that the reduced strength of the concrete would still be sufficient and precautions were taken only to secure the original quality of concrete for further work. On completion of the structure the story in question was subjected to an exhaustive test. This indicated clearly much lower results than in the case of other parts of the structure; however the results were not low enough to necessitate rebuilding so that the contractor came through with a bad scare only. If the result of the first test had been much lower, it would have been possible to tear down the one story in time. This would have been a small expense compared with rebuilding the whole structure.

In several cases of completed buildings the removal of forms was delayed time and time again in the hope that the concrete would harden sufficiently, which, however, it did not do. Such are very exceptional cases and may occur in only one of a thousand structures. One case, however, should be sufficient to warn the contractor and to demonstrate the necessity of concrete control. The extra expense is so little that it need hardly be considered in determining the cost of a structure and offers insurance against a disaster which may cause the ruin of the contractor's business.

In order to demonstrate the necessity of concrete control reference to such exceptional cases is not required. There is to be considered in first line the many small things affecting the manu-

facture of concrete which to this time have not received sufficient attention. Their importance is much under-estimated and it is consequently ignored that the combined action of these conditions may have a very bad result. In the same way as it has been customary to rely upon cement meeting standard specifications; one has also relied on the richness of the mixture, although the incorrectness of the assumption that the strength of concrete is proportional to the amount of cement can easily be proven. As soon as the mixture of sand and aggregate has reached the point where its voids are fully filled by the cement no material increase in the strength is to be expected. The extremely rich mixtures called for in many specifications are valueless and an unnecessary waste of material. It is without doubt the business of the contractor to determine the most suitable mixture of the materials at his disposal, work which, in large cities, requires occasional tests. It is clear, therefore, that the contractor must keep control of the concrete if he desires to use the most economical proportions, consistency and mixing of the materials available. The owner, *i. e.*, his engineer, or the city building inspector must also have certain information along these lines so that no requirements will be specified which it will be impossible for the contractor to meet. From several cases it can be proven that the required compressive strength in official specifications for a certain proportion of mixture cannot be obtained with the material available or the reverse, that is, the exclusion of certain kinds of stone is technically entirely unjustified. It is one of the worst assumptions that one has intuition as to the strength of the material. This judgment can only be obtained through the use for many years of such tests as are described later.

Let it be assumed that the proper preparations in a special case have been made and that the strength in compression at 8 and 28 days of the mixture of the selected proportions is known under normal conditions. The one strength serves as the control for the concrete; the other for the removal of forms. Now the question is, which extraordinary conditions on the work should be considered. In the first case the most natural deviation from the conditions originally stated would be the amount of water added to the mixture and the procedure of mixing itself. The compressive strengths determined on the test beams repeat them-

selves uniformly, which is not usually the case with concrete cubes. This is then confirmation that no mistake has been made in the manufacture of the concrete and that the completion of the work represented by the test pieces can be anticipated without worry. Even small deviations are a warning for more carefulness and checking of the methods of making the concrete.

An entirely different picture is brought to us when there is a question of the influence of abnormally high or low temperatures. In the first case it is possible that the hardening of the cement will be so rapid that concrete partially set is placed in the forms. In the latter case the action of frost can withdraw the heat necessary for the setting of the cement and destroy the concrete through the

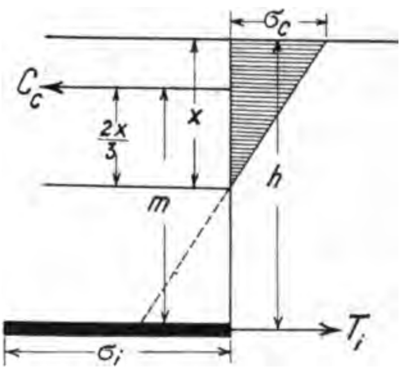

FIG. 1.

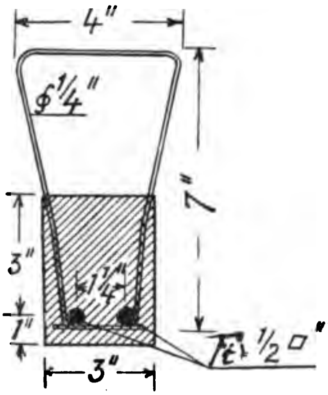

FIG. 2.—CROSS-SECTION OF CONTROL BEAM.

formation of ice crystals. These two possibilities cannot be reproduced in the laboratory. The contractor or the inspector must be able to determine on the job just what is happening to the concrete. The test pieces must be stored near the part of the structure, the concrete in which is to be controlled and must be of the identical concrete used in that part of the structure which it is desired to check up. In order to make this possible it is necessary to use a method which can be carried through right on the job without much trouble and without a testing machine.

In line with the thoughts given above, a short description of the control beam used by the author covering all the points

noted is appended. The dimensions of the test beam are so selected that the test piece will not be too heavy and can, therefore, easily be handled, and so that no large load will be necessary for the test. Further it has been borne in mind that the compressive strength should be determinable by a simple calculation from the breaking strength without complicated formulæ. This empirical rule changes according to the method of calculation, which should naturally be the same as that used in calculating the beams in the structure so that the stresses obtained can be compared and the factor of safety directly determined.

In the following the usual method of calculation is used, which

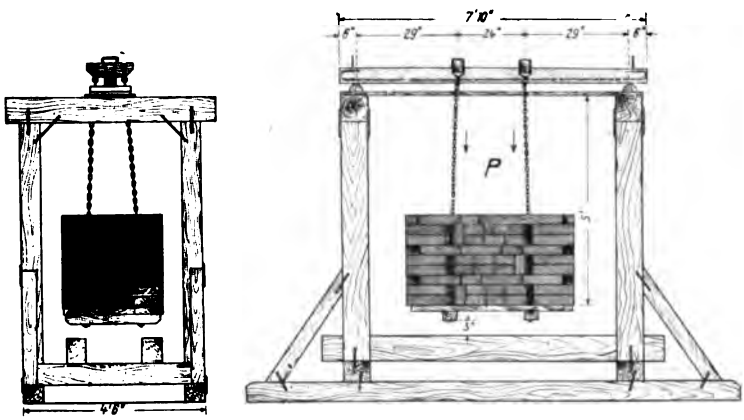

FIG. 3.—ARRANGEMENT OF TESTING APPARATUS.

assumes the rectilinear relation of the stress and deformation and the ratio of the coefficient of elasticity of the reinforcement and concrete, $n=15$, although this is a secondary detail and not directly connected with the method of testing. The following equation gives the strength of the concrete, the maximum bending moment being M.

$$M = C_c m = T_i m = \sigma_c W_c = \frac{\sigma_c}{2} x\, b\, m$$

Insofar as the notation is not shown in Fig. 1, h represents the total height, b the width of the beam, and x the distance between the neutral axis and the extreme fibre at the section in question. According to Fig. 2 in our tests the values of b and h are each 3 in

Therefore, the effective cross-section $F=3x3=9$ sq. in. and the effective section of reinforcement $F=\frac{1}{2}$ sq. in., and consequently the percentage of reinforcement is 5.55.

The arrangement of the testing apparatus, Fig. 3, gives

$$M=\frac{P}{2}\times 29=14.5\,P\ in.-lb.$$

Calculation shows

$$x=2.1\ in.\ and\ m=h-\frac{x}{3}=2.3\ in.$$

FIG. 4.—METHOD OF MAKING CONTROL BEAMS AT VIENNA.

Therefore the extreme fibre stress

$$\sigma_c=\frac{29\,P}{2\times 3\times 2.1\times 2.3}=P$$

The value of P, the breaking load, is represented by the weight of the carrying device and the bricks and $\frac{2}{3}$ of the dead weight of the beam (exactly 10.25/14.5). Having determined the load P under which the beam has failed this figure represents the value of σ_c in in.-lb. and is the desired measure of quality of the concrete tested.

Further details of the method can be obtained from *Forscherheft XIII*, "Eine Güteprobe für Beton" by Ing. G. Neumann,

as well as the author's paper presented in Berlin, 1911, to the higher officials of the building departments, Session II, published by W. Ernest und Sohn, Berlin. A description of an extensive application of the method in the construction of K. K. Kriegsministerialgebäude, Vienna, by Oberleutnant J. Kromus, appeared in *Beton und Eisen*, 1911, Heft XIX and 1912, Heft I. Figs. 4 and 5 illustrate the method of making and testing the test beams on this work.

In order to obtain correct results in such a method of test it is only necessary to be careful in maintaining constant the values

FIG. 5.—CONTROL BEAM UNDER TEST.

of F_i, the sectional area of reinforcement, in this case $\frac{1}{2}$ sq. in., and of h, the distance between top of beam and center of reinforcement, in this case 3 in. Everything else can be done so primitively as would be the case without especial care on a job and it would not affect the correctness of the results. According to experience the results obtained are always more correct, reliable and more quickly obtained than results from test cubes broken in a hydraulic press. This method of testing represents an easy way for every contractor to become acquainted with his materials and to determine whether he is obtaining the best results at the utmost economy.

DISCUSSION.

Mr. Arthur N. Talbot.—It would seem that if a satisfactory control test piece can be found, its use would be helpful to the constructor; it would serve as a record of the construction. There is a difficulty, however, in getting a beam or test piece which is fully satisfactory. I have used a plain concrete beam of a little larger cross-section than mentioned here, 6 in. wide, 8 in. deep and 36 in. in span. This of course will break in tension. It is subject to the variations which we must expect in the tensile strength of concrete and requires that more than one test piece be made, as a single result might be abnormal and accidental. Mr. Emperger has put in reinforcement in such a way as to make practically a compression test, the beam being so long that there is little danger of shear failure. Whether this form of test piece, of the size shown, would be satisfactory in practice can be told through experience with it. It would seem to me, looking at the dimensions, that a test piece only 3 in. wide and 3 in. down to the reinforcement would require such care in its construction that it would not be suitable when the aggregate is of other than very small size and that there would be objection to the size in practical use. **Mr. Talbot.**

President Humphrey.—It is a matter of interest to this Association to have this subject come up, as it is in line with the endeavor of one of our committees to have recognized the necessity of having some inexpensive field method by which the quality of concrete can be obtained. I have seen this control beam test made in Vienna and I feel that the care required to make this test renders it impracticable. The section of the test piece itself had a material bearing on the quality of the concrete and the size of the aggregate also affected in a large measure the results obtained. I did not feel from the number of tests that I saw made that there was any great concurrence in consecutive results. It is a question whether or not the same degree of accuracy cannot be obtained from a small cylinder or test piece made in a simple compression machine. Certain it is that contractors **Mr. Humphrey.**

Mr. Humphrey. should try various methods of testing, with the hope that some simple, inexpensive method for determining the quality of the concrete may be devised. That to my mind is of the most vital interest at the present time in reinforced concrete construction.

Mr. Brett. MR. ALLEN BRETT.—While as some claim the control beam in the 3 by 4-in. section might not be accurate enough to determine correctly the value of the concrete, yet, especially in running concrete in cold weather, it seems to me that the control beam would be sufficiently accurate to indicate when the concrete was hard enough to pull the forms. I think this point should be more emphasized.

REPORT OF COMMITTEE ON TREATMENT OF CONCRETE SURFACES.

The Committee has this year devoted itself to perfecting specifications for stucco and to methods of testing compounds for dampproofing of surfaces. Every member of the Association who was known to have knowledge on these subjects was written to, something over four hundred persons being addressed. There was no real criticism from anyone, and we therefore feel justified in recommending this specification for adoption this year.

The specification as printed in Volume VII of the Proceedings, 1911, should be changed as follows:

Page 587, near bottom, before paragraph "Sheathing Boards," insert:

"Frame. . The frame of building shall be so rigidly constructed as to avoid cracking the stucco."

Page 589, near bottom, change title from "Brick or Tile" to "Brick, Tile, or Cement Blocks."

Page 590, line 7, at end add "and thoroughly wetted."

Page 590, paragraph 6, change title to "Brick, Tile, or Cement Block."

Page 590, near bottom, after title "Intermediate Coat," add a sentence, "Intermediate coat may be omitted on brick, tile, or cement blocks."

Page 591, near middle, change title "Final Coat" to "Finish Coat."

Page 591, near bottom, change title "Finish" to "Surface Finish."

Page 593, after paragraph 3 add:

MACHINE STUCCO.

Stucco may be applied by a machine provided the results obtained are equal to those produced by hand work.

The last report of the Committee is amended by the following:

Insert, page 579, near bottom, under V. Waterproofing.

(*g*) *Method of testing dampproof compounds and coatings.*

Introduction—As it is obviously not the province of this Association to stoop to commercialism by publishing the names and merits of any article of trade, your Committee below gives the method by which anyone may make comparative tests of any compound for himself.

In the tests of concrete waterproofers and dampproof concrete coatings, these materials are divided into the following classes:

(1) *Colorless Dampproof Coatings.*—This class includes all preparations intended for surface application for the purpose of excluding dampness, which the makers claim will not change the appearance of the surface to which they are applied.

(2) *Black Waterproof Coatings.*—This class includes all tar, pitch, and asphalt preparations and other black waterproofing paints. Such coatings should be able to withstand some pressure head of water and are therefore termed waterproofers. They would not be used where a decorative effect is required.

(3) *Integral Waterproofers.*—This class includes all materials for waterproofing, whether pastes, powders or solutions, that are incorporated into the mass of the concrete at the time of mixing.

(4) *Colored and White Concrete Coatings.*—This class includes all coatings of a decorative nature intended for exterior use. Many of the paints in this class are recommended by the manufacturer for their dampproofing qualities.

METHOD OF TESTING DAMPPROOFING AND WATERPROOFING COATINGS.

This test is intended primarily for classes 1 and 2, although many paints in class 4, when especially recommended by the manufacturer as a dampproofer, are included.

The test piece is a 3 to 1 sand-cement mortar block 3½ in. square by 2 in. deep, with a depression in the top 2 ½ in. in diameter by about ¾ in. deep made by inverting a telephone bell in the mold. The mold is of wood, ⅞ in. oak, and is made in two right-angular pieces which hook together at opposite corners, so that it is easily removed by unhooking and drawing apart.

To obtain a porous block or one that will absorb 30 c. c. of water in more than one and less than two minutes, the sand

and cement is gauged with from 9 to 10 per cent. of water by weight (this per cent. being figured on the total weight of sand plus cement) and tamped evenly into the mold. Some difficulty may be found in making blocks of the proper porosity; that is, just dense enough to absorb the 30 c. c. of water in not less than one or more than two minutes; but after some experimenting with different percentages of water and different degrees of tamping, the proper result is obtained.

Denser blocks are made in the same manner, but the per cent. of gauging water is higher and may run up as high as 20 per cent.

All blocks are kept in a damp closet for 24 hours and then allowed to dry out in air for at least one week before being used.

After the block has dried out, 30 c. c. of water are placed in the depression and time required for the block to absorb this amount of water is noted. The blocks are divided into two classes: those which will absorb 30 c. c. of water in more than one minute and less then two, (these are called standard blocks), and those which take less than one or more than two minutes to absorb the 30 c. c. of water.

Those blocks which take up the water in less than one minute are considered too porous for a proper test.

After this treatment the block is again thoroughly dried out. The time required to absorb the 30 c. c. of water is recorded on the block which is now ready for use.

The blocks which take up water in one or two minutes are considered the standard block, but materials are tested on both porous and dense blocks for comparison.

Each block is given a number for purpose of identification. Porous or standard blocks are numbered consecutively from 1 to 500 and the denser ones from 500 up.

The waterproofing coating is applied only to the surface of the depression and to the top surface of the block. Two full, liberal and thoroughly brushed in coats are applied, the first one being given at least two days to dry out before application of the second.

Not less than one week after date of application of the second coat, the block is ready for the first test.

TWENTY-FOUR HOUR ABSORPTION TEST.

The block is weighed dry. Thirty (30) grams of water are placed in the depression. The block with the water in is allowed to stand for 24 hours, with a watch glass over the depression, after which time it is weighed again. The loss in weight is the evaporation. The water is then thrown out and the block wiped dry and weighed again. The loss from the weight, including water, is the absorption. The following is a tabulation of the diferent weighings taken:

(1) Weight of block dry plus watch glass.

(2) Weight of block plus 30 grams of water plus watch glass.

(3) Same as (2), but after standing for 24 hours.

(4) Weight of block plus watch glass, after water has been thrown out and the block wiped dry.

Weighings:

(2) Minus (3) gives weight of water lost by evaporation.

(3) Minus (4) gives weight of water left in block after 24 hours.

The watch glass almost completely prevents evaporation from the surface of the water. Before any appreciable amount of the water in the depression can evaporate it must pass through the waterproof coating and be evaporated from the surface of the sides or bottom of the block.

The most efficiently waterproofed blocks are those which show the minimum amount of absorption and evaporation.

TEST TO DETERMINE TIME REQUIRED FOR COATED BLOCK TO ABSORB THIRTY CUBIC CENTIMETERS OF WATER.

The 30 c. c. of water are placed in the depression in the block and covered with a watch glass to prevent evaporation. A paper is placed over the whole to exclude air currents. The block is examined from time to time until all the water in the depression has disappeared. The time required for the block to absorb the 30 c. c. of water is recorded. If all the water has not been absorbed in 3 weeks time the approximate per cent. absorbed in 3 weeks is recorded.

The length of time taken for the water to be absorbed by the block is an indication of the efficiency of the waterproofing film or coating.

The block is then exposed in the open air to find the effect of weather exposure on the coating. It is brought in at intervals of about 3, 6 and 12 months and the same tests as above described repeated. The decrease, if any, in the efficiency of the coating is noted after each repeated exposure.

TEST OF INTEGRAL WATERPROOFING.

For this test, the blocks are made up of a 1 cement, 2 sand mixture plus the waterproofing compound, which may be paste, powder or liquid. The waterproofing compound is incorporated in the mix in the proper amount and according to the directions as given by the manufacturer. The mixture is gauged, in all cases, with the proper amount of water to give approximate maximum density. The consistency resulting from this amount of water gives a rather wet, quaking mortar, but one from which no free water will rise to the surface on tamping.

This block is identical in size and shape to that described above in the "Test of Waterproofing and Dampproofing Coatings."

It is kept in the damp closet for 24 hours after making and is allowed to dry out in air for at least one week before being tested.

The tests on these blocks are the 24-hour absorption and time to absorb 30 c. c. of water tests and are identical with those used for the waterproofing and dampproofing coating blocks, described above in detail.

DESCRIPTION OF TEST OF CONCRETE COATINGS.

This class comprises, chiefly, coatings of a decorative nature, intended for exterior use.

The paints in this class are tested on the outer face of hollow cubes, the faces of which are about 9 in. square, with walls from 1 in. to 1½ in. thick. These hollow cubes are made of a 1 cement, 2½ sand, 4 gravel mixture; and gauged to a rather wet consistency. The block is made by packing this rather wet mixture into a 9 in. by 9 in. by 9 in. wooden form provided with an iron core about 12 in.

high and 6 in. square. The bottom of the block is made about $1\frac{1}{2}$ in. thick, the hollow iron core is then placed on this bottom layer and centered up and the concrete packed in between the core and the wooden form.

The core used has considerable taper so that it may be easily drawn out after the concrete has begun to set up. After the block has had 48 hours to harden up the wooden form is removed and the faces are finished with a 1 cement, 2 sand mortar. In finishing the faces the mortar is mixed rather wet and is merely rubbed into the surface with a wooden float to give a smooth surface. The block is ready for use not less than one week after date of finishing up the sides.

Before applying any paints, the block is given a number, marked on the inside of one of the faces, and the faces are numbered from 1 to 4, marked at the middle of the top edge. The date of making the block is recorded.

A few minutes before applying a paint, the bottom third of the face to be painted is thoroughly dampened by applying water to it with a brush. Care is taken to wait until no visible water is on the surface before the paint is applied.

In applying the first coat the paint is thoroughly brushed in so that it will penetrate all small irregularities of the surface. The second coat is applied not less than two days after date of application of the first.

At the time of painting the fact is recorded that the bottom third of the side was dampened. The following data are also noted at this time:

Number of block.
Number of side.
Name of maker and name of paint (for both 1st and 2d coats).
Date of painting (for both 1st and 2d coats).
Per cent. of sediment in the paint can when it was opened and consistency of this sediment.
Ease of application of paint.
Ease of mixing paint.
Hiding power of paint.

After the second coat has thoroughly dried, the general appearance of the painted surface is recorded.

The points noted in regard to appearance are as follows:—
Dull or glossy.
Whether surface is smooth or granular.
Whether brush marks are visible or not.
Whether paint fills well or poorly, *i. e.*, whether the irregularities of the concrete surface are well fitted up by the paint or not.

After all four sides of a block have been painted (two coats) and at least three days after the application of the final coat, the painted surfaces are scrubbed lightly with a medium bristle scrubbing brush with powdered soap and water. The effect of this scrubbing is noted in detail, as to whether the coating is softened, or whether the paint scrubs off, and if so, to what extent.

After the scrubbing test the block is allowed to dry out and after not less than one day from time of scrubbing it is filled with water and kept filled for a period of 5 days. If the water leaks through either the sides or bottom of the block to any appreciable extent, the inside of the block is given a neat cement wash to remedy this condition.

After the water has been in the block for a period of 5 days it is emptied out and the condition of the coatings on the different faces carefully examined and recorded.

Conditions resulting from the 5 days' water test to be noted are as follows:

Softening of coatings.
Excrescence.
Discoloration of coating.
Cracking or blistering of coating.
Flaking off of coating.

After the block has thoroughly dried out the coatings of the four faces are given a general comparative rating according to their respective general appearance and condition.

The block is now ready for the weather exposure test, and for this purpose it is exposed in the open air. The block is placed with the bottom side up. Examinations of the blocks are made at regular intervals of about 3, 6, and 12 months.

Conditions resulting from weather exposure to be noted are as follows:

Chalking of coatings.
Checking and cracking of coating.
Scaling or flaking off of coating.
Fading or discoloring.

Page 574. After line 8 insert the following:

A method of waterproofing has been submitted by one member of the Association and is reported to have given satisfaction. It is as follows:

For a flat surface lay a base of concrete while still wet and plaster with ⅛ in. of neat cement troweled hard, then follow with another layer of finishing concrete, the lower layer being at least 2 in. thick and the top layer 3 in.

On wall surfaces as soon as the forms are removed, thoroughly wet surface, trowel on ⅛ in. of neat cement, and follow immediately with 1 in. of 1 : 2 mortar before the neat cement has begun to dry or appreciably set.

If the wall or floor treatment in this manner is large it must be reinforced to prevent cracking.

Respectfully submitted,

L. C. WASON, *Chairman*,
CLOYD M. CHAPMAN,
ALFRED HOPKINS,
EMILE G. PERROT,
HENRY H. QUIMBY.

DISCUSSION.

The President.—There has been request for a discussion of the cement gun.* The principle of the machine is to deposit sand and cement with compressed air, the manner of the deposition being such as to obtain a density and compactness not possible by hand methods. The sand and cement are first mixed dry. It is one of the essentials that the mixture shall be dry, otherwise it will cake in the hopper or so clog up the pipe that it will not flow. The President.

The character of the surface on which it is applied and its preparation are of course the things which determine the permanency of the plaster applied. With this brief statement we may begin the discussion by requesting Mr. Chapman to say a word on the cement gun.

Mr. Cloyd M. Chapman.—The Westinghouse, Church Kerr Company conducted some rather exhaustive tests last summer on the product of the cement gun, that is the qualities possessed by mortar applied by the gun were investigated. The tests covered compressive strength, tensile strength, percentage of voids, absorption through the surface, and adhesion to other materials. In each of the tests the qualities were much superior in the gun mixed mortars than in the best hand made mortars made under similar conditions and of the same mixtures. The method was to apply a 1-in. coating on the wall. The wall was constructed with sheet metal, so hand plaster and gun plaster could be applied to the wall an inch thick. A briquette which would fit the regular tension machine was cut out with a die or cutter. For compression tests a 2-in. thick coating of mortar was cut into cubes with a die or cutter. The results of 28 different tests were from 25 to 100 per cent better in the case of the gun mixture than the hand mixture. The same applies to density measured by absorption, surface absorption, absorption through the surface film as well as total absorption on boiling 4 hours and then cooling. It showed a very dense, high quality of mortar and the uniformity was much greater than that of hand work. Mr. Chapman.

* For additional information see *Proceedings*, Vol. VII, p. 504.—Ed.

Mr. Chapman. The test pieces checked up closer and the leanness of the mix that could be used was very much greater. Some beach sand was obtained near the laboratory at Whitestone Landing, a very high percentage of which would pass an 80-mesh screen; the bulk of it, I think, would lie between 80 and 100 mesh. Proportions of 1 : 6, 1 : 8 and 1 : 10 of that very fine sand gave pretty solid mortar. No tests were made—and I want to emphasize this—of the operation of the gun as a machine.

Mr. Brett. MR. ALLEN BRETT. —It is thought that the product of the cement gun is automatically exact. It has been observed that although placing into the receiving chamber all proportions of materials, as 1 : 4 and 1 : 6 or 1 : 10, that the product on the wall is approximately always the same. There always has to be a certain amount of cement in the product on the wall and the rest of the sand drops down. It is a very peculiar proposition. The mortar on the wall always seems to have a certain amount of cement, approximately 1 : 2½. Another point is the amount of material required. The amount of material was estimated for a certain job the same as for an ordinary stucco job. The sand and cement were sent accordingly, but the material was all gone before the job was half done. It takes about twice as much material as in hand stucco or plaster. Of course the material is there as the mortar is denser.

Mr. Chapman. MR. CHAPMAN.—One point was not brought out in regard to the spatter and the failure of sand to adhere when lean mixtures were used. We did not find that the spatter was a much greater per cent of the total amount applied when lean mixtures were used as against rich mixtures, but it was greater when coarse sands were used. The trouble is caused by the larger grains which have a momentum sufficient to rebound when the mixture is not rich enough. Lean mixtures of coarse sand would rebound somewhat more than a rich mixture of coarse sand, and the rich mixture would rebound somewhat less than the lean mixture of the same sand whether fine or coarse. A rich mortar forms a putty-like surface to which the particles will adhere more easily than in the case of a lean mixture.

The President. THE PRESIDENT.—The first stage in the application of the mortar is a rebound of the sand until a sufficient coating of neat cement is deposited on the surface in which the sand will bed.

MR. CHAPMAN.—That explains the strong adhesion of gun applied mortar. The process gives a coating that next to the surface is quite rich. Mr. Chapman.

MR. WILLIS WHITED.—There are a very large number of bridges in the State of Pennsylvania, the masonry of which is built of inferior stone, which has suffered from the effects of the weather; and other bodies of masonry, some of them quite extensive, have suffered from other causes, such as bad foundations, bad mortar, etc. To repair them with stone would spoil their appearance. My idea was that the defective parts could be replaced with concrete, and then the whole faced up with the cement gun, so that they looked like concrete, and put in good repair for very much less money than it would cost to tear the whole wall down and rebuild it, thus making the whole wall better, both in quality and appearance, than it was when first built. Besides, much of this masonry was built in early times when lime mortar was used exclusively. My understanding is that mortar from a cement gun can be forced into open joints quite a distance and do a good deal better work than any repointing could, and some of the masonry could be saved in this manner that, otherwise, would have to be torn down and rebuilt. I would like to find out whether my ideas are correct. Mr. Whited.

THE PRESIDENT.—There is no doubt that the pressure under which the mortar is applied drives it into all the cracks, so as to completely fill them. With the stone in proper condition the mortar will stick to the surface, and with a first-class plasterer who should always accompany the cement gun to finish the surface as fast as the mortar is applied, a very smooth, pleasing surface can be obtained. The gun can be applied a second time to rough up, because the gun produces a pebbly surface, which prevents hair cracks and gives a pleasing finish. The President.

The great difficulty would probably be the removal of all organic growth, otherwise the mortar would not have sufficient bond and the frost would lift it off. The gun can be used as a sand blast, to clean the surface first before applying the mortar. I should think as a general proposition that the cost of reparing those bridges would, even with the cement gun, be materially less than tearing them down and replacing them entirely.

Mr. Heidenreich.

MR. E. LEE HEIDENREICH.—During the early days of the cement gun I had occasion to investigate the same for the New York Central Railroad at Grand Central Station in New York. We borrowed a gun from the manufacturer and experimented to ascertain how to cover structural steel with the gun. The greatest trouble was in losing the material going past the edges; the necessary thickness could not be obtained around the edges. There was a great waste of material, although several kinds of backing were placed behind the steel members; naturally considerable waste would occur as the mortar would pass the side of the steel member. Of course, at that time the gun was in a very incomplete condition and, I understand, it has been improved since. I would like to know if this waste of material in covering structural steel with mortar for fireproofing purposes has been diminished by some method.

The President.

THE PRESIDENT.—As far as the Chair knows the loss continues to be approximately about the same. It is interesting to note that the material collected is clean sand free from cement, which can be used again. The actual amount of material that rebounds from the surface on which the mortar is being applied remains the same.

Mr. Heidenreich.

MR. HEIDENREICH.—Taking for instance a wooden trough to stop the stream of cement mortar, which passes the steel member of course a certain amount of the charge of the gun passes down along the trough to the bottom. Suppose I want to get to the edge of a column or flange. More than one-half goes entirely by the column, and that contains cement; it sets very rapidly, too. The waste was fully one-half the total mortar material, including what fell down between the flanges, along the edge of the columns and material that passed by. This was one reason, at first, that the cement gun was not considered economical for the purpose. I do not know whether it was used since at the Grand Central Station, but this was nearly two years ago.

The President.

THE PRESIDENT.—I am not quite sure what the conclusions of the New York Central Railroad officials were on that work, but certainly the mortar was driven into all the little cracks and crevices of the iron work and produced a tightness which could not be obtained by any other method. The conditions in the New York Central Station were unusual on account of the

extreme rapidity of corrosion of the steel. Some of the steel work, erected not over two years, had rust on it to the extent of $\frac{1}{32}$ to $\frac{1}{16}$ of an inch, and the use of some protective coating to prevent further corrosion was a matter of great importance. The matter of cost was not as important to the engineers as the question of the efficiency of the coating. **The President.**

MR. CHAPMAN.—As to the sand which rebounds from the cement gun, an analysis of two or three samples of the rebounding material showed about one part of cement to twenty parts sand, although a 1 : 3 mixture was applied. Samples of the material gathered up at the foot of the wall analyzed about 1 : 20. **Mr. Chapman.**

CEMENT COATINGS.

By F. J. Morse.*

The subject of cement coatings, or cement paints, as many speak and think of them, is one which will receive a great deal more of thought. As an example, in an advertisement after describing a cement building from a fireproof standpoint, there is added, "Painting not Necessary." This remark, as it stands and as it is construed by the majority of users of cement, is correct and really applies to oil painting as it is used for the protection of lumber in building of frame construction. Good cement work does not need any protection from rain or climatic conditions such as soon destroy unpainted wood.

The difference between what is called painting or paint, or cement paint and what cement coating really is and should be, is that paint, *i. e.*, an oil paint, when applied is only a film that lies on the surface of the material applied to and has for a vehicle linseed oil. Linseed oil is useless as a covering for cement or lime because the alkali, acids and lime in cement, stone and brick surfaces, and in the air, immediately attack the linseed oil. After a year or two, at best, the life is gone and there is only a film left, that under a microscope would be found porous, moisture going through it. Water and moisture come in contact with the lime underneath and the film cracks and peels, so of course painting is not necessary on cement, but absolutely useless. But something is necessary, absolutely necessary for use on exterior cement work. Large concrete structures are veneered with brick or tile and lose their identity as cement buildings. Stucco homes are built and after a short time the surfaces crack, sometimes large patches fall away, they become very discolored; as the surface is porous oxides from the air are carried in by the rain, dirt as well; water goes through and corrodes the metal lath, and many times entirely disintegrates it. On wood lath, moisture swells them, nails rust, and the same trouble is had, cracking surfaces.

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* Heath and Milligan Manufacturing Company, Chicago, Ill.

The first thing perhaps that comes to mind would be to waterproof the cement so as to prevent the moisture and rain from going in, discoloring the work, corroding the metal lath and swelling of wood lath. This is the very thing cement coatings are made for and they not only are waterproof but add with it the decoration feature, in that most any color can be obtained in several shades for selection from natural cement colors to green,

FIG. 1.—ROUGH CEMENT STUCCO RESIDENCE COATED WHITE.

Was originally non-waterproof, like adjoining house and now looks like a white cement house; does not show painted effect.

yellow and reds. Many attractive color schemes can be worked out, especially on stucco homes.

From the fact that cement coatings are applied with a brush, like unto an oil paint on wood, many of them are called cement paints and the majority of people among users and manufacturers of cement, call it paint, so it is plain that either cement does need painting, or cement coating must not be called paint. A real cement coating must be so made and applied so that there is

penetration into pores of the surface it is applied to and not leave a film, otherwise one reverts right back to an oil paint.

Cement coatings are in a measure going through an experimental stage, by both manufacturer and user, for some of the harder requirements made of them, and too, some cement surfaces will not take a cement coating. For instance, some cement floors that are so hard and crystal-like that it is impossible to scratch them, while others are soft and porous. Now on the soft, porous floor a cement coating will harden and make waterproof and prolong the wearing of the surface a very great deal. On the hard surface that is not necessary, but the man who has this hard floor wants some uniform or decorative color to it, as it may be in a public corridor, dining room, etc. Unless the surface is treated to make it porous so that cement coating could be made to adhere, it would be useless to put it on because it would soon wear off. This can be done by using a 10 per cent muriatic acid solution as a wash to open up the surface so the coating can get a chance to adhere and penetrate.

There have been cement floors so soft that the owners of the building would not accept them and wanted the contractor to take up an inch or two and lay a new floor (a very expensive job) that have been made hard and dust proof by two coats of cement coating. There has been one coat applied to an exposed brick wall for 7 or 8 years, protecting the brick and mortar joint, still in good condition. There have been swimming pools made watertight and enameled with cement coating. A finishing coat of enamel in colors to match can be applied where enamel surface is desired, and here is a point too of much interest. A cement brick wall can be laid up and enameled with two coats of white coating and one coat of white cement enamel at a cost of about $6 per thousand brick, or $5 per square of 100 sq. ft., adding the cost of the brick, say at $10 per thousand, there is a saving of about $50 per thousand over the regular enameled brick, and at the same time the joints are as well as brick. There have been reservoirs holding 600,000 gallons of water, made waterproof by two coats of cement coating, on the inside.

To cite an example of waterproofing done with cement coating, one large corporation had built four big cement tanks, measuring about 300 ft. around. They were square, 20 ft. deep, to store

soft coal in, filling the tanks with water to keep the coal from air slacking. The walls and floor after a time cracked and would not hold the water. It was impossible to make a superficial application on the inside surface, such as the membranous method of waterproofing because the coal was taken out by a large grab bucket or clam shell, and it would break the surface; it was impossible to fill the cracks with cement, because to fill perpendicular

FIG. 2.—ROUGH CEMENT STUCCO OR PEBBLE WASH HOUSE, KENMORE AVENUE, CHICAGO, ILL.

Cracks filled and entire surface covered with olive green cement coating. Color is permanent and surface waterproof.

cracks with cement in a plaster form stiff enough to hold itself did not have sufficient water for proper crystalization and bonding, and would crumble and fall out, and grouted in, it would of course run out. These tanks however were waterproofed with cement coating, cracks filled with the coating in a heavy consistency before being reduced and afterwards all surfaces covered with the coating applied with a brush.

Another feature of cement coating is, that the cracks in a stucco house can be filled in the same manner by the use of a syringe with a spout about the size of the crack, the entire surface afterwards being coated. Cement coatings are all classed and described about alike, in that they are made for the waterproofing and decoration of cement, stone and brick and stucco surfaces and to make cement floors dustproof and stainless.

The word waterproofing covers a multitude of sins, and about the first question is, will a cement coating waterproof a basement wall on the inside to stop the water coming through the wall? The answer is a very emphatic "no," as it is not made for that purpose, no more than Portland cement is for filling teeth, or mending chairs or china. Cement coating is to waterproof only when the pressure is against it with the wall to back it up, for the exterior walls above grade, against rains and snows and moisture from the air and for decoration.

Some two years ago a rough pebble dash house was found so discolored and cracked that the owner was unable to sell or rent. The surface was so porous that the moisture had corroded the metal lath and the iron oxide showed up on the surface. This house has all been coated, cracks filled, and made pure white and the surface made waterproof so it does not show stains or discoloration.

In a recent popular article on concrete homes there is described a house of cement plaster on Long Island Sound built by one who spared no expense, and the surface is described as cracked and apparently moth eaten. Another house near by is one that had been erected by a small property owner and is of solid concrete, described as of a not very inviting color or texture. Such examples can be seen in any of the suburban towns surrounding the large cities, and is a further example of the necessity of more educational work among the builders of cement buildings.

Cement in itself is naturally absorptive and in all building work, where there is an exposed cement surface, not only from a waterproofing standpoint but from a decorative point as well, something is necessary. That something is a waterproof coating such as a cement coating. Those who have studied the situation carefully, noting the requirements, have been able to obtain many excellent results.

Cement coating can be applied to cement surfaces, including labor and material at about the following costs. On stucco or pebble dash surfaces $2.50 per square for the first coat; about $2 per square for the second coat. On sanded finished surfaces, such as interior cement plastered walls or exterior, this work can be done for approximately $2 per square for the first coat and $1.50 per square for the second coat. On cement floors one

FIG. 3.—CEMENT PLASTER RESIDENCE, EVANSTON, ILL.

All cracks filled and entire surface made waterproof and uniform in color with natural cement color coating.

coat can be applied for about $1.75 per square for the first coat and $1.25 for the second coat. Two coats are recommended. There are, however, many surfaces where one coat is sufficient, as one coat of a real cement coating will waterproof and harden the surface, but will naturally flat in differently in places according to the porosity of the surface to which it is applied. The second coat will bring the surface to a uniform color.

From the figures given above it can be readily seen that the

ordinary stucco house, which is so universally liked and which generally is of a size of approximately 35 to 40 squares, can be coated with cement coating for about $200, a very small additional expense, as compared with the satisfying results that can be obtained.

In the above article porosity is referred to as a mark of poor concrete, the result of either poor cement or of wrong proportions

FIG. 4.—CEMENT STUCCO RESIDENCE AND CONCRETE SWIMMING POOL.

House covered with cement coating and swimming pool with white cement coating and enamel.

in mixing. There is no question but that to a big extent this is true, not so much as to the statement of poor cement for having been identified personally with the cement business, I feel pretty sure that all cements that are allowed to be shipped from the mill have passed rigid inspection, and are good. Improper mixing and lack of sufficient cement is, in a great measure the cause of the necessity of waterproofing. A cement manufacturer recently said that if a greater proportion of cement were used in

all work there would be less need of waterproofing. Another thing necessary is more careful superintendence of work, as the very first principle of waterproofing is density and this can only be obtained by putting into the concrete sufficient quantity of fine material to fill the voids in the aggregate. The better concrete and cement work is done, the less need there is for cement coatings, from a waterproofing standpoint, but cement coatings

FIG. 5.—CONCRETE FIRE PRESSURE WATER TANK, 600,000 GALLONS CAPACITY. Made watertight with cement coating applied on the inside.

still add with the waterproofing feature the decorative feature, as has been explained.

A beautiful white cement house can be had by applying pure white cement coating with a brush to a cement house that has been discolored and also it can be made soft shades of green or buff, all with lime proof colors and permanent.

In applying cement coatings it is important that the surface is free from dirt and grease and that it is porous, and that the

cement coating is used and applied in a thin enough consistency to penetrate into the work. Cement coatings make cement work more pleasing to look at, and at the same time protect it from atmospheric conditions that soon plays havoc with it, especially plaster and block work.

DISCUSSION.

Mr. L. C. Wason.—Mr. Morse spoke of waterproofing tanks. I would like to know what pressure, how high a head of water it withstood. Mr. Wason.

Mr. F. J. Morse.—This tank is one built for a fire pressure tank for the International Harvester Company, holding about 6,000 gallons of water and 28 ft. in depth. They were not able to keep a good pressure. That is the only test that we have had on cement coatings, except in tanks used for storing coal. Mr. Morse.

Mr. C. W. Boynton.—It seems to me that some of the cases referred to by Mr. Morse should be explained more fully. I do not believe that poor stucco work can be saved by painting over the surface with a cement coating. Proper workmanship and proper materials must be put into structures and one should be careful not to be misled to believe that something beautiful can be made out of stucco work that is fundamentally bad. Mr. Boynton.

The commercial success of any cement coating is probably dependent very largely upon poor workmanship in concrete construction, or failure on the part of the builder to get the satisfactory results and pleasing effects which can be obtained in many ways without the use of such material. I do not want to be understood as taking a position against all coatings, waterproofing materials, etc., but I do feel that as cement users we should try to make the best possible use of the material with which we are working. Lack of knowledge as to how to handle concrete and mortar, lack of interest in trying to learn, and lack of business energy and foresight among cement users have resulted in something like sixty-five waterproofing and coating compounds for cement work being put on the market in the last four years. It takes money to keep these businesses going. The cement user and his customers are furnishing it. Just as soon as we learn to get anything like as much out of the materials with which we are working, as there is in them, this waste of money will stop. I believe, especially for the coatings, there is a field, but it must be developed along the line of imparting to concrete and mortar work some property or

Mr. Boynton. element of beauty which is not attained by the efficient handling of the materials. It is not surprising that the coating and waterproofing business as a whole is looked upon very largely as a parasite, for its development has depended to a great extent upon poor work, and so long as poor work continues, the demand for these things will continue; also the fact that poor work many times can be temporarily covered up by the use of such materials tends to encourage their use. This Association stands for quality of work and that is what we all should strive for.

Regarding the treatment of floors, there are many poor floors and we have not made much headway in solving the problem. In time we will understand the reason for dusting and will be able to write a specification, which, if followed, will assure a dustless floor. I base this statement on the one fact that dustless floors are laid today. I am confident that when men who are actually laying floors become interested and carefully note the conditions which produce dusting, and those which prevail when dusting does not result, we will be nearing the time when dusty floors are a thing of the past. I am glad to know that until such a time there are treatments which will keep down the dust, or possibly make the floor dustless.

Mr. Morse told us that we could not use a grout to repair cracks in a concrete wall because it would run through. Certainly his coating must be near the consistency of a grout, yet it seems to stay. I believe that if mortar had been mixed to the proper consistency and applied in the proper manner it would have stopped the cracks just as effectively.

Another thing Mr. Morse referred to was the making of denser concrete by the use of fine aggregates. The government is about to publish a bulletin (Technologic Paper 3, Bureau of Standards) which shows that the densest concrete is obtained with aggregates containing a very small amount of fine particles.

Doubtless Mr. Morse's coating and other coatings have their place, which is in decorating a well built structure, and not in covering up the defects in a poorly built one.

Mr. Morse. MR. MORSE.—I think I can answer in just a few words, as stated in the paper, that if better concrete was made, or better superintendence, there would be very little use for cement coatings. Cement coating is not for good cement work; it is for cement

work that is not made good. I think I tried to leave that impression with you. **Mr. Morse.**

Referring to the remarks about a part of a stucco house falling out. It was my understanding that stucco work was pebble dash work, as we know, which contracts and expands. It is a fact that the moisture beating through a stucco wall that is not waterproof, continually keeps up a corrosion of the metal lath, and the same entirely disintegrates. It is at such places where the wall falls out, and I do claim that a waterproof cement coating applied to that wall would prevent expansion and contraction to a large extent and the corroding of the metal lath.

Another question raised was filling the cracks with cement. A perpendicular crack with no bottom, a crack in any concrete wall, for example, or stucco wall, it is impossible to grout with cement. The cement will keep running out. Using a plastic cement stiff enough to hold, in setting the two sides of the cement will withdraw or dehydrate, taking the water out of this cement and not enough water remains for proper crystallization. That has been my personal experience. The cement coating is of a very heavy consistency, and put into a perpendicular crack will harden there perhaps better than cement. In this particular work cement had been tried and failed, and the grouting was a success with cement coating.

The question of taking care of cement floors is one that has caused a great deal of trouble. With a hard floor that is very crystalline, you refuse to put anything on it, but on a floor that dusts up it must be covered.

The placing a cement floor that does not dust is to my mind—we are all looking for information in this work—almost impossible, especially for a man to guarantee or assure a floor will not dust. There is a psychological moment to trowel a cement floor which can only be determined in the laboratory, and that is immediately after the initial set has taken place. The action of troweling a cement floor forms a suction and that draws the fine particles to the surface. Now if the troweling is done after the initial set has taken place it breaks up the crystallization being formed and dusting of the surface is probable.

As to the outside cement surface in reinforced concrete buildings, I have seen some work that has been rubbed down with

Mr. Morse. carborundum stone in panel effects that are beautiful, and it would be a crime to apply a cement coating to them.

Mr. Boynton. MR. C. W. BOYNTON.—I think Mr. Morse covered the whole thing when he said cement coatings are not necessary in good work. I most heartily agree with him. This Association, if it stands for anything, stands for good work. It is exactly what we are aiming at and what we are coming to. However, I am glad that Mr. Morse and his company stand ready to throw themselves into the breach and help reclaim the poor jobs.

The President. THE PRESIDENT.—There is probably no more important field than the subject of a color of some kind that can be applied to a concrete surface so as to be permanent. Most colors are more or less injured by the cement itself. They leach out so that after a very short time the color is gone. The Germans have a way of treating the concrete with an acid wash, which neutralizes the surface, and on this surface durable color can be placed. The dampness that is often found on the interior of concrete walls is in large measure overcome, and I have never seen damp walls on the interior of foreign buildings. They are generally insulated in some way with air spaces, so that the dampness with which we are troubled in a great many cases is lacking in those structures. Certain it is that the treatment of the interior so as to produce pleasing color effects is one of the most important steps that must be considered.

It is perhaps easier to tint the interior of a structure than it is to tint the exterior, and some experiments which have been made by the Committee on Treatments of Concrete Surfaces have shown that the various pigments, when they are put out in the weather, soon lose their color and coherence and practically become worthless. So that the whole subject of the treatment of the material, the conditions to be observed in applying color to concrete, is very important and very difficult.

The tendency to use selected aggregates as a color and to secure color effects through the material itself, I think, is probably the most hopeful field. The use of white cement and an aggregate which possesses color value will give tints that cannot be duplicated with ordinary pigments. Perhaps the best example of color effect is to be found in the Connecticut Avenue Bridge in Washington, D. C., where the Potomac River gravel concrete of

buff color is in contrast with the concrete which is made out of the gneiss rock of bluish color found on the site. The spandrel walls, etc., in buff are in contrast to the archrings and piers in a light blue color; and the color is distinctly visible for a considerable distance. These colors of course are permanent. The most permanent colors, as far as the exterior exposure is concerned, are due to an aggregate which is of itself permanent. That is, red can be obtained by granites and some forms of hard sandstone, which give us a permanent color effect, blue from traps and limestones, yellow from sandstones and gravels. The President.

The great difficulty in securing color effects in concrete and mortar is that the cement itself has a muddy color, and perhaps the sand is also of a dirty color, resulting in a mottled appearance, because it is difficult to add enough coloring material to overcome the muddiness produced by the sand and cement. The use of white cement and white or light sand makes it possible to add a very small percentage of color or to use an aggregate with color and secure the desired color effect. The study of this subject and the grading of the material and selecting it with the view to securing color effect is one of the developments which will characterize our future work.

REVIEW OF THE PRESENT STATUS OF IRON ORE CEMENT.

By P. H. Bates.*

It is generally understood that there are three essential constituents in Portland cement, namely: lime, alumina, and silica. It has been shown experimentally that the lime can be entirely replaced by strontium or barium, the alumina in part by iron or chromium oxide, and hydraulic materials of considerable value be obtained; but none of these have been suggested as being of commercial value with the exception of iron oxide.

There exist very extensive deposits of low grade ores in which the percentages of iron are too low to admit of their economical reduction in the blast furnaces. There are also numerous deposits of high iron oxide clays in which the high percentage of this constituent reduces the value of the clay. Both of these have appealed to the cement manufacturer but to a slight degree, owing largely to abundant supplies of clays of low iron content, from which it is comparatively easy to produce cement of normal composition. The manufacture of a cement of high iron content presents some difficulties in view of the low fusion point of the raw material, there being very readily produced the so-called—from the point of view of the manufacturer—overburned cement.

There had been noticed in several localities abroad, the failure of concrete made from cement of normal composition when placed in sea water. Whether this was due to faulty cement is an unsettled question. But various authorities on casting about for an explanation of these failures quite unanimously came to the conclusion that the trouble was due to the alumina in the cement. This constituent combines with the sulphuric anhydride of the sea water and the lime, freed in the process of the setting, to form a compound of indefinite composition. This latter crystallizes with a large water content, which pushes apart the cement particles and ultimately destroys the structure.

Necessarily the first remedy which suggested itself was the

* Chemist, Bureau of Standards, Pittsburgh, Pa.

replacement of the alumina with some material which would not have this property. Michealis, reviving the original suggestion of Malaguiti and Durocher, called particular attention to the replacement of the alumina with iron oxide. Attracted by Michaelis' arguments, the Krupps in 1901 took patents covering the manufacture of such a material.

There is no doubt but that the alumina present in cement will form a compound with the sulphuric anhydride radicle of soluble sulphates and with lime hydrate. But the exact nature of this compound is still much in doubt; just as doubtful is the question whether the compound can result from the setting of all the alumina constituents which may be present in cement. It is very generally agreed, however, that the iron compounds which are present in a cement cannot form such a decomposition product, but whether the iron is combined in the cement in a manner similar to the alumina is a very doubtful matter. The latter problem is complicated by the impossibility of securing materials for investigation which are so sufficiently free of alumina as not to be decidedly influenced by the presence of small quantities of the latter. Thus the Geophysical Laboratory has shown that 0.5 per cent. of alumina present in a silica lime mix of such a composition as to give tri-calcic silicate, will give the maximum yield of this compound. It is quite likely that those who claim the iron compounds in cement have hydraulic properties, have been misled by the influence exerted by the very small quantities of alumina which have been present in these materials. Without doubt the iron compounds lack hydraulic properties entirely, and the possibility of making a cement entirely lacking alumina will never be attained. The iron ore cements must have alumina present or else they are of no value, and in the finished cement the iron oxide is of no other value than any other material which might be present which would reduce the alumina content. This statement does not deny the value of the iron oxide as a flux in the manufacture.

It is therefore not surprising that the manufacture of this class of cement has not grown, even in Germany, as was hoped for. Its use, even in sea water, has not kept pace with the growing use of the "Eisenportlandzement" (or "Slag-Portland" cement) or the use of natural slag (Puzzuolana) in connection with Portland cement. The theory of the use of the latter is based on the fact

that the lime which is set free in the setting of the cement, will combine with the silica of the slag or puzzuolana, thereby rendering less lime available for combining with the sulphuric anhydride radicle of the soluble sulphates, and consequently less possibility of the forming of the injurious sulpho-aluminate of lime.

Notwithstanding the discussion and notices which this class of cements has received in the technical journals, there is little authentic data available showing its relative superiority over cement of normal composition. Thus, Michaelis, Jr., quotes from the *Zentralblatt der Bauverwaltung* in the *Cement and Engineering News* of March, 1911, the following compressive strengths of an iron oxide and Portland Cement in the form of 16-inch cubes:

Parts of Cement.	Parts of Sand and	Crushed Stone.	Compressive strength, (lb. per sq. in.).
1 Iron ore...............	2	3	5775
1 " "	2	3	5475
1 " "	4	6	3120
1 " "	4	6	2910
1 Portland...............	2	3	4590
1 "	2	3	5145
1 "	4	6	2415
1 "	4	6	3270

The results were obtained by the Department of Public Works Berlin at Swinemünde and represent the strength after three years immersion in sea water. While Michaelis concluded from these results that the iron ore cements are the stronger after such treatment, these conclusions are hardly justified, since the above may represent the relative strength before treatment. However, the description of test pieces, both at this point and at other localities along the German sea coast, undoubtedly shows the superiority of low alumina cements.

Quite a number of tests of iron ore cements are given in the *Tonindustrie-Zeitung* (1906, page 1968). The comparison of them with normal composition cements was carried out entirely by the use of small prismatic test pieces which were placed in the sea water and their appearance noted. Another series was conducted in which different amounts of plaster were added to the cements from which the prisms were made. In every case the resistance of the

iron ore cements to the sea water was greater than that of the Portland cements. Unfortunately, comparison by means of tensile or compressive strengths was not made. E. Maynard in the same journal (1910, page 651) conducted somewhat similar tests; he however showed the slight change in chemical composition which the iron ore cement underwent by such treatment, but does not give results for Portland cement.

In France there has been little investigative work carried on; however, Candlot* gives some interesting results obtained with five high iron oxide cements manufactured by himself. The partial chemical analyses and the strength of the neat and 1 to 3 sand briquettes at the end of the 7 and 28 day and 5 year periods, after immersion in water, are:

	No. 1.	No. 2.	No. 3.	No. 4.	No. 5.
SiO_2	19.0	21.0	26.0	24.5	25.0
Al_2O_3	5.8	5.3	5.3	5.3	5.4
Fe_2O_3	7.2	6.2	8.8	7.2	5.6
CaO	66.5	65.0	59.0	61.8	62.5
	98.5	97.5	99.0	98.8	98.5

Tensile strength, pounds per square inch.

		No. 1.	No. 2.	No. 3.	No. 4.	No. 5.
Neat Briquettes	7 days	951	845	142	220	426
	28 "	1,036	913	213	568	724
	5 years	653	824	894	769	717
1:3 Sand	7 days	355	511	92	234	355
	28 "	398	525	199	398	497
	5 years	578	710	646	724	667 (strength at end of 3 yrs.)

The author calls attention to the high lime of number 1 and mentions that the pats in steam disintegrated, although the briquettes showed no such signs; number 2 has a silica and lime content corresponding to a cement of normal composition; both of them show the retrogression in strength of the neat cement briquettes, after the 28-day period, so characteristic of cements of normal alumina content. The three other cements are really high silica cements and show the characteristics of this class very

* Le Ciment, May, 1911, page 32.

strikingly in the tensile strength—namely the very low initial strength followed by satisfactory gains.

The effect of increasing the silica content with corresponding decrease of the lime content is shown by numbers 1 and 3, and 2 and 5. Numbers 3, 4 and 5, which show decreasing amounts of iron oxide, show increasing strength at the early periods. However, these three cements show too much variation of the other constituents to attribute this gain to decreasing iron oxide.

In this same paper Candlot gives the partial chemical analyses and tensile strength, at early periods, of some silicious cements which he manufactured.

	No. 1.	No. 2.	No. 3.	No. 4.
SiO_2	26.0	25.0	27.0	26.0
Al_2O_3	2.5	1.7	3.4	2.0
CaO	67.0	66.0	65.0	67.0
	95.5	92.7	95.7	95.0
Undetermined	4.5	7.3	4.3	5.0

Tensile strength, pounds per square inch.

		No. 1.	No. 2.	No. 3.	No. 4.
Neat Briquettes	7 days	899	894	692	777
	28 "	965	909	885	866
1:3 Sand	7 days	601	398	426	440
Briquettes	28 "	667	535	586	535

These strengths hardly correspond to those of such high silica low alumina cements. In view of the large percentage of undetermined shown in the analyses, it would appear as if there were present some other constituent, not determined, which had caused these abnormally high strengths. In the discussion following the reading of the paper, Candlot acknowledged the use of a flux—"generally iron oxide"—in their preparation. We have here consequently a group of high silica, relatively high iron ore cements, which are of much interest.

No information is given in regard to the superiority of these several cements of Candlot's over normal Portland cements, when immersed in sea water. They do seem to show the high strength which it is possible to reach with high silica cements and usually the steady gains in strength. These high silicious cements

should be receiving more attention by the investigator, the manufacturer and the consumer. That Portland cement is overlimed and has principally but the ease of manufacture and ease of handling by the consumer to recommend it, is a growing conviction. The latter statement is substantiated by the increasing use of trass, tufa, etc., which use would be impossible were not normal Portland cement overlimed.

Until within a very recent period there has been no attempt in this country to manufacture a high iron Portland cement and place it on the market as such. There have been on the market for a number of years, however, two cements, manufactured here, which are of exceptional high iron oxide content, although this fact has not been used by their manufacturers for recommending them for other than the ordinary purposes. In the two cements referred to the percentage of iron oxide is little less than the alumina.

There has been just as much inattention in this country devoted to the investigating of this class of cement as there has been to its manufacture. About a year* ago there was conducted a series of tests at the University of Illinois in which it was desired to show the superiority of this class over that of normal composition cements for sea water use. The high iron cement used was manufactured in the Ceramic Department of the University, and was compared with a normal Portland cement secured on the market. According to the tests reported, which consisted of making pats and briquettes of the two cements and treating with sea water under pressure and observing the disintegration and loss of strength, the former was superior to the latter. But other burnings of high iron oxide cement made here gave the opposite results; the author, however, explains this by improper composition of his burnings, so that on the whole his results are not very conclusive. He however did find that alumina was absolutely essential; a burning in which it was entirely replaced by iron oxide did not give a material which could be called a hydraulic cement.

The only other extensive series of tests made in this country are those which were carried out at the Atlantic City Laboratory of the Technologic Branch of the U. S. Geological Survey, now a part of the Bureau of Standards.

In Table I are given some of the results obtained, using not

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* See p. 597, ED.

TABLE I.—TENSILE STRENGTH IN POUNDS PER SQUARE INCH OF NEAT AND 1 : 3 SAND BRIQUETTES OF VARIOUS CEMENTS WHEN IMMERSED IN FRESH AND SEA WATER.

	Slag Cement (E).		Natural Cement (I).			
	Neat Briquettes.		Neat Briquettes.		Sand Briquettes.	
	Fresh Water.	Sea Water.	Fresh Water.	Sea Water.	Fresh Water.	Sea Water.
4 weeks......	549	686	282	302	116	111
13 "	558	730	368	368	196	165
26 "	534	652	378	236	182	99
52 "	422	432	366	283	260	208

	Mixture, Natural and Portland Cements (J).				Portland Cement (K).	
	Neat Briquettes.		Sand Briquettes.		Neat Briquettes.	
	Fresh Water.	Sea Water.	Fresh Water.	Sea Water.	Fresh Water.	Sea Water.
4 weeks......	221	258	247	190	865	1100
13 "	216	311	201	256	910	927
26 "	204	286	257	318	830	900
52 "	250	244	218	276	463	923

	Portland Cement (L).		Portland Cement (M).	
	Neat Briquettes.		Neat Briquettes.	
	Fresh Water.	Sea Water.	Fresh Water.	Sea Water.
4 weeks............	595	623	748	837
13 "	557	507	630	799
26 "	550	673	610	570
52 "	407	233	507	277

	Slag Cement (F).			
	Neat Briquettes.		Sand Briquettes.	
	Fresh Water.	Sea Water.	Fresh Water.	Sea Water.
4 weeks............	416	468	210	180
13 "	511	512	306	241
26 "	590	348	246	258
52 "	536	210	336	280

	German Iron Ore Cement (A). Neat Briquettes, Fresh Water.	Neat Briquettes, Sea Water.	Sand Briquettes, Fresh Water.	Sand Briquettes, Sea Water.	Special Low Alumina Portland Cement (B). Neat Briquettes, Fresh Water.	Neat Briquettes, Sea Water.	Sand Briquettes, Fresh Water.	Sand Briquettes, Sea Water.
4 weeks..........	535	702	120	134	764	714	242	252
13 "	714	746	221	166	803	554	311	355
26 "	726	802	234	236	800	622	418	320
52 "	764	656	256	204	768	748	370	310

	Portland (C). Neat Briquettes, Fresh Water.	Neat Briquettes, Sea Water.	Sand Briquettes, Fresh Water.	Sand Briquettes, Sea Water.	Typical Portland No. 47 (G). Neat Briquettes, Fresh Water.	Neat Briquettes, Sea Water.	Sand Briquettes, Fresh Water.	Sand Briquettes, Sea Water.
4 weeks..........	672	748			604	821	268	206
13 "	744	808			636	136	300	238
26 "	708	768			658	**	304	258
52 "	636	764			674	**	350	205

TYPICAL PORTLAND CEMENT NO. 47 (G).

	Specimens stored in damp closet for 24 hours; then for 2 days in fresh water. Neat Briquettes. Fresh Water.	Sea Water.	Specimens stored in damp closet for 24 hours; then for 6 days in fresh water. Neat Briquettes. Fresh Water.	Sea Water
4 weeks............	672	959	700	926
13 "	765	904	723	840
26 "	713	852	734	940
52 "	622	738	660	598

	White Portland (H). Neat Briquettes.* Fresh Water.	Neat Briquettes.* Sea Water.	Sand Briquettes. Fresh Water.	Sand Briquettes. Sea Water.	French Portland Cement (D). Neat Briquettes. Fresh Water.	Neat Briquettes. Sea Water.
4 weeks......	658	996	212	164	539	511
13 "	808	970	290	200		
26 "	742	788	366	162	702	656
52 "	630	502	292	204	776	706

* A second lot of briquettes gave substantially the same results.
** Broke in handling, before placed in testing machines.

only this class of cements but in addition for the sake of comparison, Slag, Natural, White, Slag-Portland, and normal composition cements. Among the latter is a well-known French Portland cement which while of normal composition is highly recommended and rather extensively used for sea water concrete. Its distinctive characteristics are the low sulphuric anhydride content and its decided coarseness, 21.3 per cent. being retained on the 100 sieve and 38.8 per cent. on the 200. Unless otherwise noted, all the briquettes were stored for 24 hours in the damp closet before immersion in the waters. In Table II is given the chemical analyses of all cements, the tensile strengths of which are reported in Table I.

An examination of Table I shows that the neat briquettes made of Portland cement of normal composition retrograde considerably in strength after the 13 weeks period. In this class have not been placed the cements K and C since both of them show rather high iron oxide content, although they could hardly be called iron oxide cements. Cements L and M were simply chosen at random from standard Portland cements, while the typical Portland is a mixture of equal parts of a large number of standard Portland cements. The effect of aging, even for a short period in fresh water before subjecting the cement to the action of sea water, is very closely shown, and suggests a practice which should be observed, when at all possible, in all cement used in sea water. The rather remarkable behavior of the White Portland, in view of the theory under which high iron oxide cements are used, is striking and unexplainable. The data on E has been given largely because this cement, if we are rightly informed, belongs to that class of cements called by the Germans "Eisenportland" cements, which are mixtures of slag with Portland cement, though contrary to the German practice more slag has been used in its manufacture than Portland cement. This cement, while it shows retrogression, does not show the decrease in strength which the true slag cement does.

The sand briquettes of all the classes compare rather favorably, and it would be difficult to form an opinion of the relative value of any class from the behavior of the mortars alone. With the exception of the Cement J, all the sand briquettes show less strength in sea water than in fresh water at the end of a year;

TABLE II.—CHEMICAL ANALYSIS OF CEMENTS, THE TENSILE STRENGTHS OF WHICH ARE SHOWN IN TABLE I.

	A	B	C	D	E	F	G	H	I	J	K	L	M
SiO_2	23.44	20.37	22.51	23.04	27.77	30.19	21.50	22.66	22.51	20.90	21.36	19.82	22.07
Fe_2O_3	7.48	8.97	4.86	2.51	.60	1.64	2.28	.55	4.86	2.44	3.73	2.10	2.31
Al_2O_3	2.98	3.64	5.42	7.18	13.87	11.08	8.12	8.61	5.42	6.02	6.77	7.62	6 95
CaO	61.86	61.42	61.87	63.60	44.07	46.16	62.23	62.46	61.87	32.96	63.43	62.04	62.33
MgO	.50	.82	.93	1.04	4.49	2.17	3.24	1.10	.93	21.21	1.59	3.90	2.28
SO_3	1.72	1.19	1.47	.43	1.22	1.10	1.45	1.64	1.46	2.15	1.41	1.43	1.54
Na_2O	.20	1.54	.17	.23	.42	.29	.18	.40	.17	.51	.09	.24	.31
K_2O	.28	.24	.48	.63	.11	.64	.38	.53	.46	2.53	.31	.26	.57
H_2O (-105°)	.42	.....	.....	.....	.....	.....	.33	.36	.....	.27	.43	.90	.45
CO_2	.69	1.01	.89	.27	2.10	.72	.12	.63	.89	10.17	.78	.92	.45
Ig. loss	.09	1.06	1.82	1.17	2.68	3.28	.39	1.07	1.82	.95	.36	.90	.92
CuO	.51	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
CaS	.....	.....	.....	.....	2.84	2.74	.....	.....	.....	.....	.....	.....	.....
	100.17	100.26	100.41	100.10	100.19	100.16	100.27	100.01	100.41	100.11	100.26	100.13	100.18

those of cements I, F and White at 1 year do not show a retrogression since the previous period of testing.

The relative strength in compression of concrete made from German iron ore cement and normal Portland cement is shown in Table III. All specimens were stored for 3 weeks in a damp closet before immersion in the water, and while the concrete

TABLE III.—COMPARISON OF COMPRESSIVE STRENGTHS OF CONCRETE MADE FROM AN IRON ORE AND A PORTLAND CEMENT.

GERMAN IRON ORE CEMENT (A).

Compressive strength in pounds per square inch (1 part cement, 2 parts sand, 4 parts trap rock).

Stored 8 weeks in damp closet before immersion in—

	Fresh Water.	Sea Water.
13 weeks,	3743	3477
26 "	specimen 3987 A	B
52 "	B	B

A. Two of three specimens did not break under a load of 4237 pounds, capacity of the testing machine.

B. All specimens did not break under a load of 4237 pounds, capacity of testing machine.

TYPICAL PORTLAND CEMENT No. 47 (G).

Compressive strength in pounds per square inch (1 part cement, 2 parts sand, 4 parts trap rock).

Stored 8 weeks in damp closet before immersion in—

	Fresh Water.	Sea Water.
13 weeks,	3190	3877
26 "	3457	3979
52 "	3389	4060

from the former cement is stronger than the latter, yet it is not very decidedly marked. At all periods the Portland shows greater strength in sea water than in fresh water, whereas, at the early period the Iron Ore shows a less and apparently at the later periods about equal strengths in the two waters.

These abstracts from the data of the Atlantic City Laboratory show in a general way the relation of the strengths of the Iron Ore to various other cements. Again, it cannot be positively stated

that the former show much superiority over the latter classes of cements. The information to be obtained in foreign publications is usually of a descriptive character and is concerned but little with numerical data, which of necessity are the only positive and reliable data. In this respect the information and data are much different from that obtainable concerning the "Eisenportland" cements, and if one is to judge of the use of the latter from the numerous investigations which have been and are still being carried on in regard to its general use and use in sea water concrete, it must be a very important and valuable product, far more so than the "Erzzement" or Iron Ore cement.

MARINE OR IRON ORE CEMENTS.

By Herman E. Brown.*

As soon as Portland cement became an accepted building material, engineers and chemists commenced investigations as to its behavior under the varying conditions to which it was exposed. The object of these investigations was two-fold—to assure themselves of the permanency of the structure already erected, and to find the outer limits of the field in which it could be used.

As early as 1854 Malaguti and Durocher (*Compt Rendus*, Vol. 39, p. 183) were investigating the cause of disintegration of concrete masses in sea water. As a result of such investigations on commercial hydraulic cements, and on synthetic cements manufactured by them, they state: "The inertness of the oxide of iron in the hydraulic materials appears demonstrated by these synthetic tests. We are forced to conclude that the presence of this oxide is able to enhance the stability of cement mortars in sea water."

Malaguti and Durocher had been inspired to make these investigations on account of the discoveries of Vicat in 1840, who had found that a certain hydraulic cement, after six months in sea water, had been greatly damaged, an analysis of the cement showing that the magnesium had been largely increased and the calcium oxide decreased by about the same amount.

In 1882 Landrin makes the following statement: "The calcium aluminates in cement are extremely harmful for permanency of mortars in sea water, because they dissolve easily." (*Thon Industrie Zeitung*, 1882, p. 177.) Other prominent investigators now began to take a lively interest in the search for the exact cause of the occasional sickness of concrete structures in sea water. Le Chatelier and his co-workers commenced a series of painstaking experiments. In 1890 Le Chatelier writes: "The double sulphate of calcium and aluminum plays an important

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role in the destruction of concrete in sea water." Candlot in 1892 discovered the double salt of calcium and alumina. In 1899 commissions were appointed by the German and French cement manufacturers for the purpose of studying the action of concrete in sea water. The scientific and literary heirs and assigns of the said commissions are still investigating, and sometimes reporting.

In 1892 Michaelis, in a paper read at London to the Society of Civil Engineers, made the following statements: "From the chemical point of view, cements or hydraulic limes rich in silica and as poor as possible in alumina and ferric oxide should be used, for aluminate and ferrate of lime are not only decomposed and softened rapidly by sea water, but they also give rise to the formation of double compounds, which in their turn destroy the cohesion of the mass, by producing fissures and swelling." (*A. S. C. E.*, Vol. 107, p. 375.)

Schuljatschenko, Eger, Rebuffat and Vicat did notable work on the complex questions involved in their attempts to solve the difficulty. The most convincing work was done by Le Chatelier, who, some time previous to 1900, made several synthetic cements and subjected these cements to the action of sea water. These cements were manufactured in sufficient quantities to carry out long-time tests by actual immersion of the test pieces. Portland cements were manufactured, consisting of silica, alumina and calcium, in which the alumina varied from 4.1 per cent. to 16½ per cent. Iron oxide cements were made, in which there was no alumina present, and consisting only of silica, ferric oxide and calcium oxide; in these, the percentages of iron ran as high as 14 per cent. He also manufactured cements in which the oxides of manganese, cobalt and chromium were used, as substitutes for the ordinarily occurring alumina. In 1900, 1901 and 1902 he reported before various scientific bodies on the action of sea water on these special cements. As a result of his investigations, Le Chatelier took the position that the most important, if not the only, cause of disintegration of cement in sea water is the formation of calcium sulpho aluminate, and that the iron oxide cements were much more resistant to sea water than the alumina cements. (See Reports of Paris Congress of Testing Materials, 1900; Buda Pest Congress, 1901; and *Thon Industrie Zeitung*, 1902, No. 11.)

From 1902 to the present, publications and controversies have been frequent and confusing.

In the fall of 1909, the American Cement Engineering Company erected an experimental Portland cement plant, two views of which are herewith presented—Fig. 1, a general view of the entire plant, and Fig. 2, a closer view of the kiln.

The plant consists of a rotary kiln 20 ft. by 2 ft., set at an inclination of $\frac{3}{8}$ in. per ft., revolving at 2 turns per minute. The

FIG. 1.—INTERIOR OF LABORATORY.

kiln has a stack, 1.5 ft. in diameter by 18 ft. high; thickness of kiln shell, $\frac{1}{4}$ in.; lining—Harbison-Walker Special Aluminous fire blocks, 3 in. thick. By means of counter-shafting and a variety of different sized sprockets, several changes on the speed of the kiln can be quickly made. The slowest speed revolves the kiln at such a rate that the material requires somewhat over an hour to pass from feed to discharge end. It was found that a speed which would permit of the material passing through the kiln in one-half

hour was entirely satisfactory from the standpoint of quality of clinker produced, and output. The kiln is provided with a No. 6 Rockwell fuel oil burner, complete with connections, and uses crude oil, or distillate which is sent into the atomizing line under a pressure from the pump of about 30 lb. per sq. in. A small three-piston Gould power pump was first installed, but as the valves in this pump gave considerable trouble because of residues in the oil, this pump was replaced by a single-acting piston power pump, of a capacity at least four times greater than was

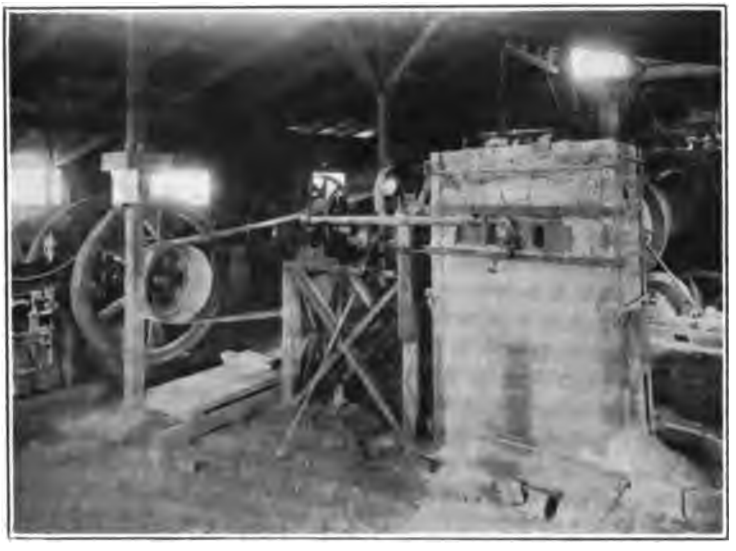

FIG. 2.—EXPERIMENTAL KILN.

required. A steady control of feed was secured by valves in the feed pipe, the excess oil returning to the reservoir. The oil used was distillate.

The blower, as shown on the left of the pump, is a Rockwell No. 1 type A positive pressure blower, provided with a relief valve, set at 2 lb. pressure.

The reduction machinery in this plant consists of:

1 Sturtevant 8 x 5 laboratory roll.
1 Sturtevant 2 x 6 laboratory crusher.

1 Sturtevant sample edge grinder.
1 Abbe No. 5. 3 ft. x 3 ft. 6 in. pebble mill.
1 Abbe No. 6. 30 in. x 30 in. pebble mill.
1 Perfecticon screen.
1 Pan dryer, with sheet iron surface 5 ft. x 12 ft.

The whole plant is operated by a 12 H.P., 200 R.P.M. Backus gasoline engine.

This plant was designed for the purpose of manufacturing cements in sufficient quantity to make any desired practical or laboratory test with the various classes of cement or raw materials which might be submitted for examination.

The ferruginous Miocene shell marls at Yorktown, Virginia, furnish excellent raw material for the manufacture of a complete series of Marine cements ranging in alumina content from 2 to 7 per cent., and in iron oxide content from 3½ to 9 per cent. Sufficient runs were produced so that practically the entire range of cements were secured, from a high ferric oxide cement to a high alumina cement. Enough was made at each run so that the characteristics of the cement in course of manufacture, the behavior of the raw materials in process of clinkering in the kiln, and the grindability of the raw and finished product and the chemical and physical qualities of the cement were ascertained.

The shell marls furnished from 75 to 100 per cent. of the raw materials entering into the various cements. When additions were necessary, clays of such analyses were selected that the resulting cement would have the desired content of ferric oxide and alumina.

On account of the fact that the low-alumina cements so manufactured are especially adapted for concrete marine structures, I would recommend the name "Marine" cements, in preference to the term "Iron ore" cements, since the latter signifies that iron ores are used in the manufacture of the cements.

The ferruginous shell marls at Yorktown, Virginia, carry sufficient amounts of lean, ferruginous clays to make a suitable mix for Marine cements, without the addition of tempering materials. In some of the runs made, however, in order to secure as high ferric oxide content as desired, pyrites cinder of the following analysis was used:

Ferric oxide	79.19
Alumina	4.73
Calcium oxide	1.82
Magnesium oxide	1.35
Silica	7.72
SO_3	4.95
Sulphur as sulfide	1.96

In the beginning of the "Iron ore" cement industry in Germany, the cements as made for commercial purposes carried less than 1½ per cent. of alumina. These cements were so slow in setting, and had physical characteristics so different from the standard Portland cements, that it was difficult to convince the buyers of their merit. It is probably on this account that the manufacturers of "Iron ore" cement in Germany have gradually

TABLE I.

Lab. No.	24 Hrs.	7 Days.		28 Days.		3 Mos.		Fineness.		Per Cent. H_2O	Temp.	Boil.	Chemical Analysis.							
	Neat.	Neat.	Sand.	Neat.	Sand.	Neat.	Sand.	100.	200.				SiO_2.	Al_2O_3.	Fe_2O_3.	CaO.	MgO.	SO_3.	Ignition Loss.	Total.
1272	*44	462	185	649	318	828	445	98	85.4	21	55	O. K. but soft	23.82	1.43	8.71	61.75	0.75	1.78	1.47	99.71
1313	174	635	265	681	370	830	393	95	78	22	70	O. K.	19.88	7.80	2.64	62.20	....	1.39	3.43	97.34
1350	255	602	197	845	358	862	455	97	83.5	21	70	O. K.	22.64	3.08	7.96	62.50	0.96	1.15	1.35	99.64

*Two days.

increased the content of alumina, since recent importations show the alumina content to run slightly in excess of 3 per cent.

Non-aluminous cements are practically impossible to manufacture, on account of the difficulties of securing raw materials free from alumina; and even if they could be commercially manufactured, are less satisfactory than the cements which carry sufficient amounts of alumina to render the resulting cement active enough to meet the demands of practical constructors, who require hydraulic cements to obtain their final setting within a reasonable time—certainly sooner than 48 hours.

Table I shows the chemical and physical characteristics of (first) an imported low-alumina "Iron ore" cement; (second) a cement from the middle west; and (third) a Marine cement made in the laboratory plant above described, the tests being

TABLE II.

Lab.	Fe_2O_3.	Al_2O_3.	SiO_2.	CaO.	Fe_2O_3 / Al_2O_3.	CaO. / Silicates.	SiO_2. / R_2O_3.	Fineness. 100.	Fineness. 200.	Setting Time. Initial.	Setting Time. Final.	Per cent. H_2O.	Temp.	Boil.	1 Day.	2 Days.	7 Days.	28 Days.	90 Days.	180 Days.	270 Days.	360 Days.	
*1272	8.71	1.43	23.82	61.75	6.1	1.818	2.25	98	85.4	8°-30′	48°	21	55°	O.K but soft	....	44	462	649	828	....	1032	974	Neat
															....	....	185	318	445	....	643	541	Sand
□1242	8.57	3.17	21.82	62.11	2.7	1.85	1.86	97.6	85	5°-30′	10° 40′	19	55°	O.K	217	....	726	885	1007	916	958	911	Neat
															....	....	265	407	532	528	537	504	Sand
1361	7.8	2.9	22.50	61.98	2.7	1.86	2.1	96	87	4°-30′	7° 0′	22	70°	O.K	219	....	699	803	879	....	....	....	Neat
															....	....	270	404	459	....	....	....	Sand
1239	6.56	3.96	23.74	63.53	1.66	1.85	2.25	97	85	3°-0′	8° 0′	21	55°	O.K	104	....	501	808	912	979	943	904	Neat
															....	....	207	388	573	549	528	511	Sand
1231	7.63	4.09	22.64	62.44	1.85	1.81	1.93	96.6	85.4	3°-0′	10° 0′	22	60°	O.K	253	....	647	812	891	....	....	....	Neat
															....	....	255	394	518	....	....	....	Sand
1319	4.46	5.70	23.04	62.85	0.78	1.89	2.26	97	84	2°-45	5° 30′	20	70°	O.K	477	....	781	972	917	....	....	....	Neat
															....	....	294	410	501	....	....	....	Sand
1191	4.75	5.89	22.34	62.70	0.80	1.90	2.10	97	89	3°-0′	5° 45′	20	70°	O.K	364	....	714	896	907	855	859	1215	Neat*
															...	....	373	463	577	567	504	715	Sand
*1314	2.45	7.81	20.90	60.99	0.31	1.95	2.03	95	75.4	4°-45′	7° 00′	22	65°	O.K	430	....	677	753	817	810	789	829	Neat†
															....	....	302	472	416	358	302	320	Sand
R20	8.12	3.62	21.72	60.32	2.24	1.74	1.85	97.6	85	5°-30′	10° 40′	19	55°	O.K	359	....	749	855	940	916	958	....	Neat
															....	....	325	500	529	528	537	....	Sand
1238	8.66	3.58	21.46	61.68	2.42	1.82	1.75	97	85	4°-30′	9° 00′	21	55°	Fair	194	....	802	970	960	....	....	....	Neat
															....	....	275	473	575	....	....	....	Sand
1265	8.38	3.26	23.10	61.33	2.57	1.76	2.00	95	86	3°-00′	6° 30′	21	60°	O.K	118	....	585	880	949	....	....	....	Neat
															....	....	234	448	571	....	....	....	Sand
1341	7.57	2.75	23.24	63 27	2.74	1.88	2.25	95	84	....	....	....	....	O.K	130	....	618	767	813	....	....	....	Neat
															....	....	266	401	448	....	....	....	Sand
1350	7.96	3.08	22.64	62.50	2.60	1.85	2.05	97	83.5	2°-0′	6° 30′	21	70°	O.K	255	....	602	845	862	....	....	....	Neat
															....	....	197	358	455	....	....	....	Sand
□1250	9.88	2.72	21.58	60.46	3.63	1.77	1.71	95.2	84.5	4°-20′	9° 40′	....	....	....	213	....	629	808	862	....	....	....	Neat
															....	....	268	466	488	....	....	....	Sand

* Air last 3 Mo. † MgO 34.3.

TABLE III.—EFFECT OF ACCELERATORS ON GERMAN IRON ORE CEMENT No. 1272.

Accelerator Used.	Initial Set.	Final Set.	Steamed 5 hrs. after 24 hrs.	Color, etc., at 24 hrs.
No. 1272. Without addition.	8°–30′	40°–00′	O. K., but no strength.	
2 per cent. Dicalcium Aluminate.	7°–00′	Just about final at 22½ hrs.	O. K. Hard.	O. K.
4 per cent. Dicalcium Aluminate	5 –00′	Hard set at 22½ hrs.	O. K. Hard.	O. K.
1 per cent. Cooking Soda.	8°–30′	Good final set at 22½ hrs.	O. K. Hard.	Slight efflorescence and white spots.
2 per cent. Cooking Soda.	25′	9°–00′	O. K. Hard.	Not very hard at 24 hrs. Bad efflorescence.
1 per cent. Caustic Soda.	2°–30′	Good final set at 21 hrs., but not hard.	O. K. Hard.	Badly frosted all over.
2 per cent. Caustic Soda.	45′	Very hard at 21 hrs.	O. K. Hard.	Very badly frosted all over.
2 per cent. Sod. Calcium Aluminate.	3°–15′	Good final set at 22½ hrs., but not hard.	Leaves glass; but hard.	Slight gray blotches.
4 per cent. Sod. Calcium Aluminate.	2°–10′	Final set at 22½ hrs., but not hard.	O. K.	Slight gray blotches.
2 per cent. Hydrated Lime.	5°–30′	"	Leaves glass whole but no strength.	O. K.
4 per cent. Hydrated Lime.	2°–30′	Final set at 22 hrs and quite hard.	"	O. K.
2 per cent. Hydrated Lime.	3°–30′	Barely set at 21½ hrs.	O. K., but no strength.	Dull dead surface.
1 per cent. Cooking Soda. 2 per cent. Hydrated Lime.	3°–45′	Very hard at 21 hrs.	Leaves glass, but O. K., hard.	Snow white.
2 per cent. Cooking Soda. 2 per cent. Hydrated Lime.	1°-30′	Very hard at 21 hrs.	O. K. Hard.	Snow white.
Lab. No. 1023. Without addition.	1°–5′	18 hrs.	O. K. Not very strong.	Good.
2 per cent. Dicalcium Aluminate.	3°–30′	6°–30′	O. K., after 2 days.	Good color.
4 per cent. Dicalcium Aluminate.	3°–20′	7°–20′	"	Good color. Firm pat.
2 per cent. Tricalcium Aluminate.	3°–30′	7°–00′	"	"
4 per cent. Tricalcium Aluminate.	3°–00′	7°–00′	"	"

carried out according to the standards of the American Society of Testing Materials.

The important differences in these cements are the relative percentages of ferric oxide, alumina, magnesia and sulphur trioxide contained in them. From an inspection of this table, it will be noted that the ferric oxide content runs as low as 2.64, and as high as 8.71 per cent., the alumina running as low as 1.43 and as high as 7.8 per cent. The magnesia in No. 1313, while not determined, is known to run about 2 per cent.

Table II gives the characteristics, under standard conditions, of a selected series of Marine Portlands. For the sake of com-

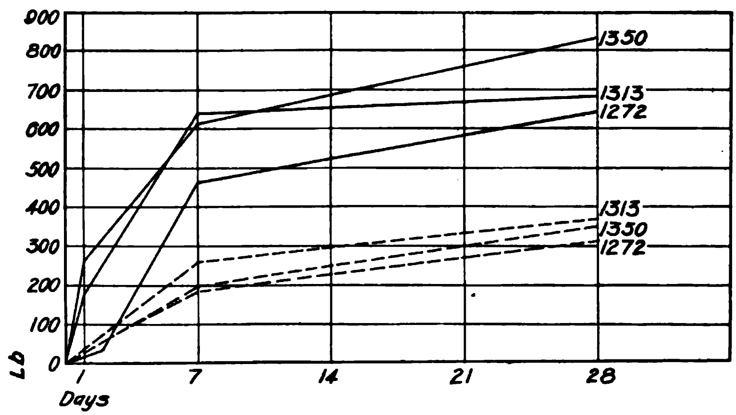

FIG. 3.—CURVES SHOWING COMPARATIVE STRENGTHS BETWEEN IMPORTED IRON ORE CEMENT AND MARINE CEMENT MADE AT YORKTOWN, VIRGINIA, AND A STANDARD PORTLAND IN THE MIDDLE WEST.

TESTS MADE ACCORDING TO A. S. T. M. STANDARDS.

parison, No. 1272 is again inserted, and a highly aluminous cement from the Lehigh Valley, No. 1314, is included.

No. 1250 gives the tests as secured by a well-known independent laboratory on cement No. 1242. This same cement (No. 1242) was sent to three different testing laboratories, and checked fairly well with the results secured in our own laboratory, so far as physical tests were concerned, there being a much wider variation in the chemical tests than in the physical.

As a consequence of the chemical differences in the cements in Table II, a striking contrast in the physical properties as re-

gards setting time and initial strengths of the cements (or what is known as the activity of the cements) is apparent. These distinctions are more graphically shown in the curves of strengths in Fig. 3, 4 and 5.

Very low-alumina cements behave on setting much like Puzzolans, and it is difficult to decide when the cement takes its initial set. The final set in "Iron ore" cements carrying under 1½ per cent. of alumina is from 40 to 50 hours. With such slow-setting cements the accelerated tests for soundness (such as boiling the pats of neat cement) should not be carried out until 1 day after final set has been secured.

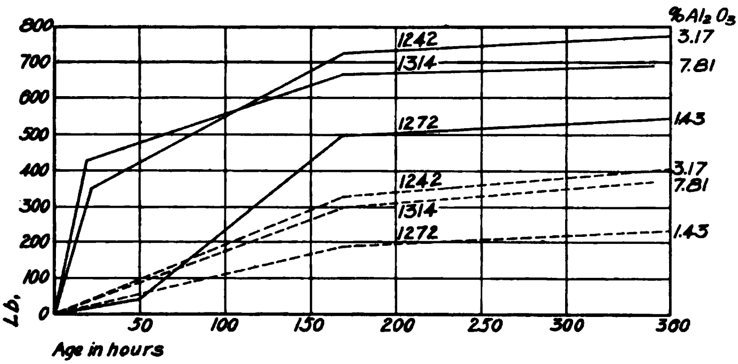

FIG. 4.—CHEMICAL ACTIVITY CURVES ON CHARACTERISTIC CEMENTS. FULL LINES NEAT. BROKEN LINES SAND.

Plaster accelerates the setting time of low-alumina cements, but if used in excess of 3 per cent. shows ill effects, the expansion being so great that the pats under boiling disintegrate completely. The probable explanation for this behavior is that the crystallizing action of the plaster starts the setting action in the cement before the free lime has time to hydrate completely.

A series of investigations was started upon these low-alumina cements, to ascertain the effect accelerators would have upon setting time, soundness and color. Numerous accelerators were mixed with imported "Iron ore" cements No. 1272 and No. 1023. Standard methods for setting time and accelerated tests for soundness were carried out. These results are shown in Table III.

The aluminates of calcium gave the most satisfactory results as accelerators for these slow-setting cements. The aluminates were prepared in a small laboratory kiln, according to the following molecular formulas:

$2\ CaO\ Al_2O_3$ for dicalcium aluminate.
$3\ CaO\ Al_2O_3$ for tricalcium aluminate.
$Na_2O\ CaO\ Al_2O_3$ for sodium calcium aluminate.

The effect of tricalcium aluminate, while not given in Table III, differs from the dicalcium silicate in that with corresponding percentages, the final set is secured with tricalcium aluminate from 30 to 45 minutes later than with the dicalcium aluminate. It would appear from the results secured on laboratory No. 1023 sample, as shown in Table III, as though the aluminate exercised no hastening action on the setting time. But the setting time recorded in the table without addition is that which was secured when the cement was first obtained. The barrel was permitted to stand exposed to the air, and after one month's time thus exposed, this cement had lost its activity. This behavior is characteristic of the very low-alumina cements that have their setting time regulated by the addition of plaster. After such loss of activity, it is impossible to decide when these cements secure their final and initial sets. The addition of aluminates to cement No. 1023 caused it to become active, and in so far as setting time and soundness were concerned, it then had the characteristics of a normal Portland cement.

Table IV shows the effect of aluminates on tensile strengths of slow-setting iron ore cements. The beneficial effect of the addition of these aluminates is most marked on the mortar briquettes, which were made according to usual methods, the proportions being 1 of cement to 3 of sand, and these proportions hold good for all of the sand briquettes described in the tables.

The tensile strengths reported in all of the tables given are averages of from three to five briquettes in every instance.

In order to carry out some accelerated tests on Marine cements, Feret's methods were followed. Neat briquettes were made up in accordance with the A. S. T. M.'s standards, and after 3 months' curing, sharp-edged prisms were ground out

of the briquettes. These test pieces were approximately $\frac{5}{8}$ in. square by $2\frac{1}{2}$ in. long, varying from 1.5 to 3 in. owing to the difficulty of sawing them from the solid neat briquette.

Galvanized sheet iron strips 18 in. by 3 in. were bent into U-shaped supports, and the prisms of neat cement tied to the supports. A galvanized sheet iron container with a tight cover was used for the solution, which was kept at all times at a height such that there was complete immersion of all the test pieces.

TABLE IV.—EFFECT OF ALUMINATES ON TENSILE STRENGTHS OF SLOW-SETTING IRON ORE CEMENTS.

IMPORTED IRON ORE CEMENT.

	Analyses.		
	Cement.	Lime.	Bauxite.
Silica	23.26	1.50	2.02
Alumina	2.09	0.50	64.55
Ferric Oxide	8.43		
Calcium Oxide	62.54	94.96	
Magnesium Oxide	0.73		
Sulphur Trioxide	1.66		
Loss by ignition	1.36	2.99	29.45

	7 days.		28 days.	
	Neats.	Sands.	Neats.	Sands.
Without additions	484	123	684	163
Addition of 4 per cent. Tricalcium Aluminate	506	233	726	379
Addition of 4 per cent. Dicalcium Aluminate	489	303	643	451

NOTE.—In manufacturing the Aluminates, the proportions taken for combination were based only on percentage content of Calcium Oxide and Alumina in the Lime and Bauxite.

Clear rain water, to which various percentages of magnesium sulphate were added from time to time, constituted the solution in which these sticks were immersed.

The first immersion was made on May 19, 1910, in a $\frac{1}{4}$ per cent. magnesium sulphate solution. This solution was changed to a $\frac{1}{2}$ per cent. solution on May 1, 1910. On May the 24th, the solution was changed for a fresh one of $\frac{1}{2}$ per cent. magnesium sulphate. Every 5 days thereafter for 5 weeks the solution was made up anew, the old solution being thrown away.

Thereafter, every week the ½ per cent. solutions were renewed, up to November 16, 1910, when the examination was made upon which Table V is based.

The numerical scale in Table V is based upon the physical

TABLE V.

Lab. No.	Description.	Condition.
1254	Standard Portland Cement from the Middle West	10. Corners all gone. Pits in faces.
1247	Standard Portland Cement from the Middle West	9. Corners all gone.
1314	Standard brand from Lehigh Valley	9. Corners all gone.
1259	A Lehigh Valley standard Portland Cement.—No. 1 Piece	2.
1259–A	A Lehigh Valley standard Portland Cement.—No. 2 Piece	Perfect.
1223	Silica—Lime Cement—Run No. 2	8.
1171	Silica—Lime Cement—Run No. 1	7.
1191	High-iron Portland—No. 1 Piece	1.
1273	High-iron Portland—No. 2 Piece	Perfect.
1239	Marine Portland	Perfect.
1231	Marine Portland	Perfect.
1265	Marine Portland	Perfect.
1242	Marine Portland	Perfect.
1272	Imported Iron ore cement	Perfect.

CHEMICAL CONSTITUENTS WHICH INFLUENCE DISINTEGRATION.

Lab. No.	Per cent. of CaO.	Per cent. of Al_2O_3.	Per cent. of SO_3.	Per cent. of MgO.
1254	62.20	7.80	1.39	n.d.
1247	61.75	7.35	1.47	n.d.
1314	60.99	7.81	1.61	3.43
1259	61.39	6.65	1.31	3.17
1191	62.70	5.89	1.21	0.98
1273	62.77	4.65	1.34	1.12
1239	63.53	3.96		0.83
1231	62.44	4.09	0.97	0.98
1265	61.33	3.26	0.82	0.93
1242	62.11	3.17	0.96	0.90
1272	61.75	1.43	1.66	0.73

appearance of the test pieces after the adhering calcium carbonate or other salts which sometimes covered the pieces had been washed from them. No. 10 was chosen for the piece showing the greatest disintegration, perfect condition being represented by No. 1.

No. 1223 and No. 1171 were special cements, manufactured by grinding pure quartz sand very finely, and then adding to it an equal amount of Portland cement, analyzing as follows:

Silica	23.12	per cent.
Alumina	6.95	"
Ferric oxide	4.65	"
Calcium oxide	60.01	"
Magnesium oxide	1.45	"
Sulphur trioxide	1.56	"

After adding the cement to the finely ground sand, 10 per cent. of thoroughly hydrated lime was added to the mixture. The whole was then ground to a fineness such that 85 per cent. would pass through a No. 200 mesh sieve, and 95 per cent. through a No. 100 mesh sieve. The cement so manufactured gave excellent long-time tests, comparing favorably with standard Portlands on neats and mortar strength, from and after the 28 day period.

The object of subjecting this special cement to the accelerated tests was to throw some light on the often-advanced theory of the benefits arising from waterproofing and silica additions to standard Portlands which are intended to be used in concrete structures in sea water. It will be noted that these cements showed nearly as much disintegration as the standard Portland cements, and were far inferior in their resisting qualities to the Marine Portlands.

From an inspection of the chemical constituents which have an influence on disintegration, it appears that the cements which are low in sulphur trioxide, alumina and magnesium oxide, withstand perfectly for the time immersed the disintegrating effects of the solution. If a comparison be made between the Lehigh Valley brand which withstood perfectly the disintegrating effects of the solution and the Lehigh Valley brand which was classed as "Nine" under "Condition," it will be seen that the magnesia, sulphur trioxide and alumina all run higher in the brand which was disintegrated than in the one which showed perfect condition.

After November 16, 1910, when the above conditions were noted, the test pieces were allowed to remain for 3 months without changing the solution. During this time most of the pieces became coated with calcium carbonate, which retarded the

disintegrating effects of the solution. On March 1, 1911, the coatings of calcium carbonate were removed by washing with weak acid, and subsequent cleansing with pure water, and the test pieces were again immersed in a 2 per cent. solution of magnesium sulphate.

In order to ascertain what effect sea water would have on the various classes o Marine and ordinary cements, comparative tests were made for a period of 9 months. The sea water at Yorktown, Virginia, was found to be practically the same as that at the mouth of the Chesapeake Bay. This sea water was evaporated so that it would occupy but one-quarter of its original volume. Care was taken to prevent evaporation of this solution, the old solution being rejected and a new one made every 3 months. Briquettes were made according to the A. S. T. M.'s standards, and after curing under a damp cloth for 24 hours, were immersed respectively in fresh water and in sea water solutions. Table VI gives the results of these tensile tests, and also the chemical analyses of the cements.

The curves deduced from the tensile strengths show at a glance the difference in behavior of these various cements, Fig. 5.

No. 1263, No. 1265 and No. 1246 are Marine cements, manufactured from the Yorktown shell marls. All of the cements subjected to this test showed to some extent the deteriorating influences of the concentrated sea water. The Marine cements, however, offered much greater resistance under the same conditions, than did cements Nos. 1313, 1314, and 1259. No. 1314 is a high-alumina cement from the Lehigh Valley district, as is also No. 1259. No. 1313 is a cement from the middle west. The 12 months' test on No. 1246 shows 876 lb. for the neat, and 324 lb. for the sand. It is evident, therefore, that the sand briquette is steadily gaining in strength from the 6 months' period, while the neat briquette shows a loss between the 9 months' and the 12 months' period.

The alumina runs practically the same in No. 1314 and No. 1313, and, as already stated, the magnesia in No. 1313, while not determined in this particular sample, runs slightly less than 2 per cent. in this cement.

No. 1259 shows much better resistance in the accelerated tests than No. 1314 (again it will be noted that No. 1259 runs

TABLE VI.—COMPARATIVE TENSILE TESTS OF VARIOUS CEMENTS IN FRESH WATER AND IN CONCENTRATED SEA WATER.

Lab. No.		24 Hours.	7 Days.		28 Days.		3 Months.		6 Months.		9 Months.		Chemical Analysis.							
			Neat.	Sand.	Neat.	Sand.	Neat.	Sand.	Neat.	Sand.	Neat.	Sand.	SiO_2.	Al_2O_3.	Fe_2O_3.	CaO.	MgO.	SO_3.	Ignition Loss.	Total.
1273	Fresh Water...	281	595	205	837	408	898	530	......	......	......	......	24.08	4.65	4.89	62.77	0.97	1.34	1.29	99.99
	Sea Water.....	......	679	173	1180	363	1138	493	1169	382	......	......								
1265	Fresh Water...	118	585	234	880	448	949	571	......	......	......	......	23.10	3.26	8.38	61.33	1.00	0.82	1.40	99.29
	Sea Water.....	......	610	215	1008	440	1193	463	1083	394	901†	360								
1246	Fresh Water...	275	800	323	1079	437	1039	487	......	......	......	......	22.56	5.68	4.00	63.80	0.95	1.10	1.46	99.55
	Sea Water.....	......	1062	308	1178	428	1061	347	1078	288	1091	303								
1314	Fresh Water...	430	677	302	753	472	817	416	810	358	......	*	20.90	7.81	2.45	60.99	3.43	1.61	2.82	100.01
	Sea Water.....	......	1003	293	1119	398	932	185	308	127	217	55								
1313	Fresh Water...	174	635	265	681	370	830	393	805	341	......	......	19.88	7.80	2.64	62.20	......	1.39	3.43	....
	Sea Water.....	......	835	247	1041	327	881	312	957	287	......	......								
1259	Fresh Water...	213	652	254	777	391	680	483	......	......	......	......	22.56	6.65	2.93	61.39	3.17	1.31	2.14	100.15
	Sea Water.....	......	815	349	886	483	1152	318	1013	297	......	......								

Fresh water tests made according to usual standard methods.

Sea water tests ditto; except after 24 hrs. they were stored in sea water boiled down to 4 concentrations.

Results given are averaged from 2 to 4 breaks.

* Two briquettes broke in clips.

† One briquette.

lower in alumina, magnesia and in sulphur trioxide than No. 1314).

As a result of these tests, it again appears that the content of alumina, calcium sulphate, and magnesia plays a very important role in the resistance which the cements are able to show against concentrated sea water.

In the manufacture of Marine cements, there are certain ideas prevalent in relation to the ease of burning, which do not

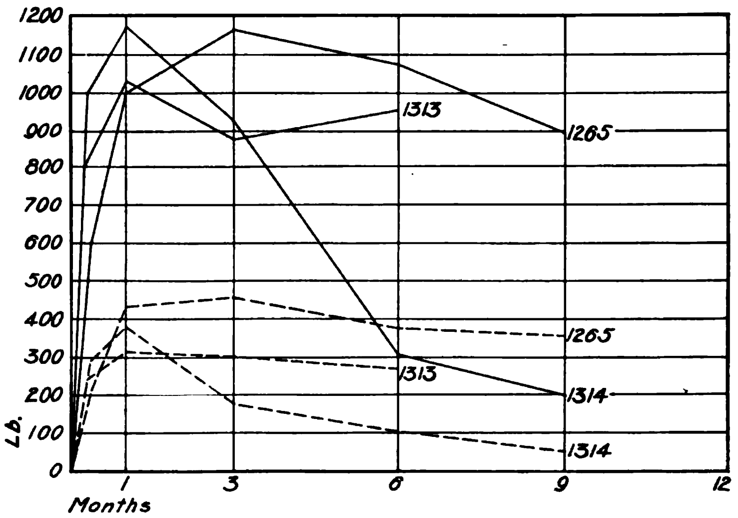

FIG. 5.—CURVES SHOWING COMPARATIVE TENSILE TESTS OF MARINE AND STANDARD PORTLAND CEMENTS IN CONCENTRATED SEA WATER.
RESULTS GIVEN ARE AVERAGES OF 3 TO 5 BRIQUETTES.
FULL LINE CURVES NEAT STRENGTHS.
BROKEN LINE CURVES SAND 3 TO 1 STRENGTHS.

seem justified when these cements are actually brought under commercial methods of manufacture. It is frequently stated that the oxides of iron are better fluxing materials for Portland cement than alumina. Experience shows that in the burning of these cements which carry high percentages of ferric oxide, combination between the basic and acidic elements does not commence at as low temperatures as with the aluminous cements. Also, the temperature interval between the combination tempera-

ture and the temperature of fluxing is much narrower in the Marine cements than in the aluminous cements. It is easy to distinguish in the kiln the increase of heat, due to the formation of the calcium silicates and calcium ferrites; and because this increase of heat is given off in a much more restricted area than occurs in the manufacture of aluminous cements, care is required to prevent fluxing.

When the raw materials are ground to the same degree of fineness, it is not possible to carry the lime to the silicates ratio as high in Marine cements as in aluminous cements, if sound cement is to be secured. This behavior indicates that ferric oxide is not a complete substitute for the alumina, and cannot carry relatively as much of it in combination, molecule for molecule, as alumina.

The percentage of allowable coarse particles, *i. e.*, particles failing to pass the No. 200 mesh sieve, is less for Marine cement raw materials than for aluminous cement raw materials, if sound cements are to be secured. No trouble, however, was encountered in the manufacture of sound Marine cements when the raw materials were ground to such a fineness that 85 per cent. would pass a No. 200 mesh sieve, and 95 per cent. a No. 100 mesh sieve. It is quite possible to make sound Marine cements, that differ only slightly in density from the aluminous cements. The low temperature interval between temperature of combination and temperature of fluxing is responsible for the density ordinarily secured in Marine cements. When these cements are manufactured with the specific gravity running two-tenths higher than the ordinary Portlands, it is found that the clinker so produced is very difficult to grind. It is advisable, therefore, from the standpoint of economy in production of Marine cements, to prevent incipient fluxing conditions, and thereby secure a cement which has only a slightly higher specific gravity than the aluminous cements.

The problem of the underlying causes for the failure of cement structures exposed to the action of sea water is so complex that it is not surprising that there are many and opposing views held by able investigators, who have spent much time and earnest effort in the attempt to solve the question.

The chemical and physical action which stands out clearly

in this work, extending over a period of nearly 2 years, can be summed up as follows: When disintegrating effects occur in concrete exposed to the action of sea water, and where there are variations in level of the water, due to wind, wave and tide, these disintegrating effects may be rightly divided into two main causes—mechanical and chemical. Mechanical action is influenced (first) by alternate drying and wetting of the surfaces, due to wind, wave and tidal action; (second) abrasions from ice or floating objects; (third) formation of expansive crystals where freezing occurs, or where double salts may be formed within the concrete body.

This brings us to the second main cause of disintegration, namely, chemical action, since the formation of the double salts is first brought about by substitutive chemical reactions. Such double salts as calcium aluminum sulphate occupy more space than the original compounds. The dissolving, replacing and formation of these new crystalline substances act in the same manner as ice crystals within the rocks. Such ice crystals have been very effective in producing disintegration of the ancient rocks covering the earth's surface.

The hydraulic cements which are best fitted to withstand satisfactorily all disintegrating effects of sea water (excepting those which are purely mechanical) are those which give the greatest density of concrete structure, and which are relatively high in silica, low in magnesia and sulphur trioxide, and in which the content of alumina does not exceed that of the ferric oxide.

In carrying out the work herein described, the author has had the constant help and valuable assistance of J. H. Payne, who was chemist for the American Cement Engineering Company, at Yorktown, Virginia, during the prosecution of these investigations.

IRON ORE CEMENT.*

By Arthur E. Williams.†

Iron ore cement is a product intended to be used in sea water work. This material is now manufactured in Europe under the name of Erz cement. According to Mr. William Michaelis, Jr.,‡ the process of manufacture is similar to that of Portland cement except that limestone and iron ore are used in place of limestone and clay. United States Consul Thackara§ gives a description of its manufacture as follows: Chalk, flintstone, and finely ground ferric oxide are used. The flint and iron are ground together, then mixed with the chalk and water and screened through a fine sieve. The screened product is clinkered in a rotary kiln and then ground. An average composition of iron ore cement, given by Michaelis is:

CaO	63.5 per cent	Al_2O_3	1.5 per cent
SiO_2	20.5 "	MgO	1.5 "
Fe_2O_3	11.0 "	Alkali	1.0 "

The effect of sea water is undoubtedly two-fold. In the first place chemical reaction may take place between certain constituents of the cement and the salts in sea water, and, on the other hand, the mechanical action of the waves carrying large amounts of sand, freezing, thawing, and the varying pressure of the water due to tide help to injure the cement submerged in sea water. This work, however, will be confined to the chemical action of sea water, for the mechanical action is of minor importance unless the cement is weakened by chemical changes.

The reactions which take place between Portland cement and sea water are said to be of three distinct kinds. *First*, the action of $MgCl_2$ and $MgSO_4$ in sea water on the calcium hydrate formed during the hardening process of the cement, forming $Mg(OH)_2$, $CaCl_2$, and $CaSO_4$. *Second*, the action of gypsum,

* Under the direction of Mr. R. T. Stull.

† Urbana, Ill. A Thesis for the Bachelor of Science Degree in Ceramics, University of Illinois in 1910.

‡ *Eng. News*, Vol. 58, pp. 645–646.

§ *United States Consular Reports*, June, 1908.

$CaSO_4$ formed above, upon the calcium aluminates forming calcium sulpho aluminate. *Third*, the crystallization of the gypsum and calcium sulpho aluminate giving an increase in volume, thus causing the disintegration of the mortar.

That free lime is present in set Portland cements is well known. Lamine* found 32 per cent of CaO in cement submerged in the Black Sea 15 years. Every analysis of a cement exposed to sea water shows a high percentage of MgO. Vicat† in 1840 showed this fact clearly, a cement, which was submerged in sea water for 6 months, was analyzed. A sample, taken from the surface exposed to the sea, showed 10.4 per cent MgO and 19.3 per cent CaO while the interior, which was not impaired, showed 1.87 per cent MgO and 31.33 per cent CaO.

A. Meyer‡ states that cement loses strength in sea water. The $MgSO_4$ acting with the silicate of lime forms $Mg(OH)_2$ and calcium sulphate. The $CaSO_4$ reacts with the calcium aluminates (Al_2O_3, x CaO) of the cement, forming $Al(OH)_3 + 3\ Mg(OH)_2 + CaSO_4 + CaCl_2$.

Charles J. Potter§ says that $MgSO_4$ is the most active constituent in sea water on cement. He found that $MgCl_2$ softens cement but causes no expansion. Potter says that it is now definitely believed that magnesium salts act on the feebly combined lime and alumina compounds which on taking up water of crystallization cause bursting of the concrete. He mixed calcined red brick clay with Portland cement clinker in proportions of 6 to 10. From this mixture briquettes were made and placed, together with Portland cement briquettes, in fresh water, sea water, and sea water to which 10 per cent $MgSO_4$ was added. Both of these cements gained strength in fresh water. In salt water, the Portland cement briquettes began to fail after 5 weeks and were disintegrated after 5 years. These cements showed blistering after one year, which was followed by expansion and bursting. The red cement improved continually but took 8 weeks to obtain the maximum strength that the Portland cement had obtained in 5 weeks. In the 10 per cent solution of $MgSO_4$, the Portland cement tested 500 lb. in a month and then went

* *Le Ciment*, 1901, pp. 111–691–81.

† *Iron Ore Cement*—The P. C. Co. of Hemmoor, Hamburg, Germany.

‡ *Chemisches Central Blatt*, Vol. 73, p. 1368.

§ *Jour. Soc. Chem. Ind.*, Vol. 28.

back to zero in 1 year. The red cement began at 250 lb. and increased continually to 1015 lb. in 8 years. Mr. Potter says that the chemical combination of CaO, SiO_2, and Al_2O_3 and water is feeble and that probably accounts for the ability of magnesium in sea water to be so active.

The experiments of Dr. Michaelis* and Le Chatelier† lead them to the conclusion that Portland cement suffers in solutions containing sulphuric acid salts, which applies to sea water. A double salt is formed composed of gypsum and calcium aluminate. This sulpho-aluminate, Al_2O_3, CaO + $3CaSO_4$, is said to crystallize with 30 molecules of water, which process must be accompanied by considerable expansion. Le Chatelier says that "the main cause if not the sole cause, of the injuries which cements suffer under the action of sea water is the formation of calcium sulpho-aluminate."

Rebuffat‡ says on the contrary that sulpho-aluminates cannot exist in cements in sea water but agrees with Michaelis and Le Chatelier that calcium aluminates are the parts of cement most easily acted upon by salts in sea water.

It has been shown that calcium ferrates are formed similarly to the calcium aluminates and that alumina could be replaced by ferric oxide in Portland cement. Dr. Michaelis puts this knowledge into use with the idea of overcoming the disintegration in sea water. The result of this application is the Iron Ore cement of today.

Dr. Michaelis and the Royal Experiment Station of Charlottenburg have tested these cements in comparison with Portland cements in a very thorough manner. Mr. William Michaelis§ says in a paper read in the United States that tests of Erz cement and Portland cement were made with both neat and 3 to 1 mixtures which were placed in fresh water, sea water, and water containing five times more salt that sea water. In sea water, the Erz cement developed a much greater strength than the Portland. In the strong salt water, the strength of the Portland cement decreased rapidly while the Erz cement showed a steady gain. Briquettes were made of Iron Ore and Portland cement

* *Thon Industrie*, 1896, p. 838.
† *Le Ciment*, 1901, p. 31–32.
‡ *Thon Industrie Zeitung*, 1901, p. 272.
§ *Eng. News*, Vol. 58, pp. 645–646.

which were placed in a salt solution of five times the normal strength of sea water under pressure of 15 atmospheres for a few days. This condition destroyed the Portland cement briquettes entirely, while the Iron Ore cement increased in strength.

The Royal Experiment Station conducted similar tests to the above but much more elaborate. Two Iron Ore and three Portland cements were made into prisms, using a 3 to 1 mixture of standard sand and cement. These prisms were placed in sea water and water containing five times the percentage of salts in ordinary sea water. In addition to this, these three solutions were allowed to act upon test pieces made of cement mixed with varied amounts of gypsum. All the Portland cement mortars disintegrated in the three- and five-fold salt solutions; all the Iron Ore cement mortars remained intact and sound.

United States Consul A. W. Thackara* investigated this cement for use on the Panama Canal. The result of his investigations was the adoption of this cement for concrete work exposed to sea water. Another point in favor of this cement is the property of slower setting. The cement is weaker than Portland for the first week, but then gradually gains strength and exceeds that of Portland.

Publications of previous experiments do not show definitely the best composition for cements giving the greatest protection against sea water. With this idea in view, the following investigations were undertaken:

The outline of procedure in these experiments is as follows: Newberry's cement formula, x $(3CaO, SiO_2) + y(2CaO, Al_2O_3)$, was used as a basis. Assuming, according to Newberry, that Fe_2O_3 could replace Al_2O_3 and form $2CaO, Fe_2O_3$, a triaxial diagram was plotted (Fig. 1), the three members stationed at the three corners being $3CaO, SiO_2$, $2CaO, Al_2O_3$ and $2CaO, Fe_2O_3$. By blending these three members, cements could be obtained containing various amounts of the calcium aluminate and the calcium ferrate.

The batch weights of these three members were calculated and about 15 kg. of each were weighed up, using practically chemically pure materials. Whiting, flint, aluminium hydrate, and red oxide of iron were the only ingredients. These batches

* *United States Consular and Trade Reports*, June, 1908.

were ground in a ball mill, then passed through a 200-mesh sieve; thus getting thorough mixing and a finely ground batch. The formulæ for the cements made are given in Table I.

The following cements, No. 19, 20, 21, 22, 23, 24, 25, 36, 37, 38, 39, 40, 42, 48, 49, 50, 51, 52, 53, 54, 58, 59, 60, 61, 62, and 65 on triaxial diagram were then weighed up, blunged thoroughly, and partially dried by pouring the slip into plaster molds.

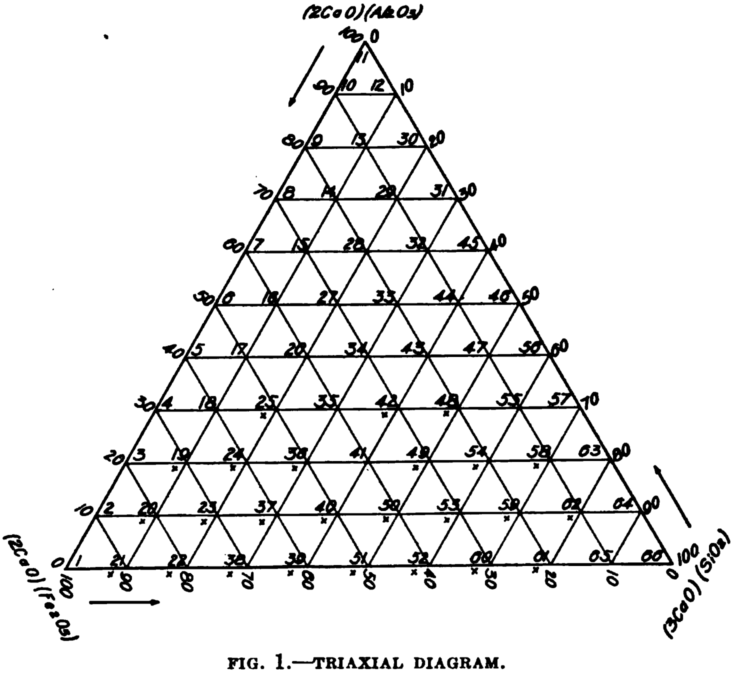

FIG. 1.—TRIAXIAL DIAGRAM.

The cements were then rolled into small balls about the size of a marble, dried, dehydrated in a down draft kiln to about 800° C. and placed in fruit jars ready for burning.

These cements were burnt in a magnesite test kiln, designed by Mr. Stull of the Ceramic Department, especially for burning experimental cements. The construction of this kiln is shown in Fig. 2. The success of this kiln is a noteworthy fact as test

kilns suitable for this purpose, heretofore, have not been very satisfactory owing to lack of control, unevenness of temperature in the clinkering chamber. Kerosene oil was used for fuel with an air pressure of about 50 lb.

The temperature at the time the clinker was drawn from the kiln was determined first by means of a Wanner pyrometer. This was given up, however, as the rapid rate of burning required a higher temperature than the true temperature of clinker formation.

TABLE I.—FORMULÆ OF CEMENTS MADE.

No.	Formulæ.	Molecular Ratio $SiO_2:AlO+Fe_2O_3$
19	$.1(3CaO,SiO_2)+.2(2CaO,Al_2O_3)+.7(2CaO,Fe_2O_3)$	0.11
20	$.1(3CaO,SiO_2)+.1(2CaO,Al_2O_3)+.8(2CaO,Fe_2O_3)$	0.11
21	$.1(3CaO,SiO_2)+.9(2CaO,Fe_2O_3)$	0.11
22	$.2(3CaO,SiO_2)+.8(2CaO,Fe_2O_3)$	0.25
23	$.2(3CaO,SiO_2)+.1(2CaO,Al_2O_3)+.7(2CaO,Fe_2O_3)$	0.25
24	$.2(3CaO,SiO_2)+.2(2CaO,Al_2O_3)+.6(2CaO,Fe_2O_3)$	0.25
25	$.2(3CaO,SiO_2)+.3(2CaO,Al_2O_3)+.5(2CaO,Fe_2O_3)$	0.25
36	$.3(3CaO,SiO_2)+.2(2CaO,Al_2O_3)+.5(2CaO,Fe_2C_3)$	0.43
37	$.3(3CaO,SiO_2)+.1(2CaO,Al_2O_3)+.6(2CaO,Fe_2O_3)$	0.43
38	$.3(3CaO,SiO_2)+.7(2CaO,Fe_2O_3)$	0.43
39	$.4(3CaO,SiO_2)+.6(2CaO,Fe_2O_3)$	0.66
40	$.4(3CaO,SiO_2)+.1(2CaO,Al_2O_3)+.5(2CaO,Fe_2O_3)$	0.66
42	$.4(3CaO,SiO_2)+.3(2CaO,Al_2O_3)+.3(2CaO,Fe_2O_3)$	0.66
48	$.5(3CaO,SiO_2)+.3(2CaO,Al_2O_3)+.2(2CaO,Fe_2O_3)$	1.00
49	$.5(3CaO,SiO_2)+.2(2CaO,Al_2O_3)+.3(2CaO,Fe_2O_3)$	1.00
50	$.5(3CaO,SiO_2)+.1(2CaO,Al_2O_3)+.4(2CaO,Fe_2O_3)$	1.00
51	$.5(3CaO,SiO_2)+.5(2CaO,Fe_2O_3)$	1.00
52	$.6(3CaO,SiO_2)+.4(2CaO,Fe_2O_3)$	1.50
53	$.6(3CaO,SiO_2)+.1(2CaO,Al_2O_3)+.3(2CaO,Fe_2O_3)$	1.50
54	$.6(3CaO,SiO_2)+.2(2CaO,Al_2O_3)+.2(2CaO,Fe_2O_3)$	1.50
58	$.7(5CaO,SiO_2)+.2(2CaO,Al_2O_3)+.1(2CaO,Fe_2O_3)$	2.33
59	$.7(3CaO,SiO_2)+.1(2CaO,Al_2O_3)+.2(2CaO,Fe_2O_3)$	2.33
60	$.7(3CaO,SiO_2)+.3(2CaO,Fe_2O_3)$	2.33
61	$.8(3CaO,SiO_2)+.2(2CaO,Fe_2O_3)$	4.00
62	$.8(3CaO,SiO_2)+.1(2CaO,Al_2O_3)+.1(2CaO,Fe_2O_3)$	4.00
65	$.9(3CaO,SiO_2)+.1(2CaO,Fe_2O_3)$	9.00

Almost all of these cements were fused till the surface was glassy in appearance before the cement seemed well clinkered and crystals appeared. Cements No. 54, 58, 62, and 65 appeared like a Portland clinker, except darker in color and were not fused or slag-like in appearance.

The clinker was first reduced in a jaw crusher and then ground in a disc mill; a screen test showed 24.2 per cent on 150 mesh screen; 12.3 per cent on 200 mesh screen; and the remainder, 63.5 per cent passed 200 mesh. These cements show that they are approximately of the same degree of fineness as the average Portlands. After the samples were ground, pats were made from

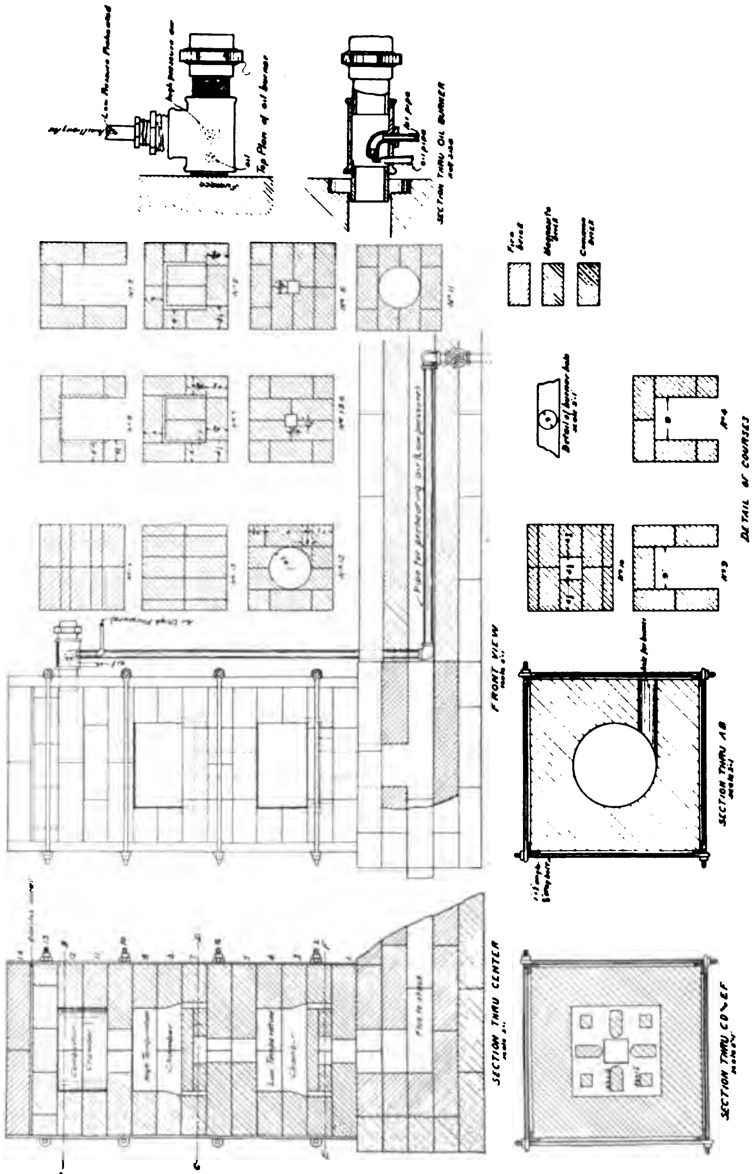

FIG. 2.—HIGH TEMPERATURE FURNACE FOR BURNING CEMENTS.

them in the usual manner to determine the properties of the cement.

The amount of water used for mortar was determined by the Boulonge method (Waterbury's Cement Manual, p. 44). The initial and final sets were determined with Gilmore needles.

Four pats were made of each cement with the idea of using one for the time of setting tests and placing the other three immediately in the moist closet, two of which were to be used for the boiling test after 24 hours, the third to be allowed to stand in

TABLE II.—RESULTS OF TESTS ON CEMENTS.

No.	Time of Initial Set, hours.	Time of Final Set, hours.	Water Used, per cent.	Remarks at Time of Final Set.	Conditions after 48 Hours in Moist Closet.
19	1½	3	21.0	Cracked in ½ hour	Cracked
20	1	5	20.0	O. K. Strong	Warped and cracked
21	2½	5½	21.0	No cracks	No cracks
22	1⅔	4	20.0	Small cracks	No cracks
23	1	..	21.0	Cracked	No cracks
24	¾	5½	22.0	Cracked	No cracks
25	1¼	..	21.5	Cracked	No cracks. Soft
36	1¾	11	20.0	Cracked	O. K.
37	1	2½	20.0	Cracked	No cracks
38	1	5	20.0	O. K.	No cracks
39	2	8	21.0	Cracked	Cracked
40	1¼	3½	20.0	Cracked	Warped
42	½	2¾	21.5	O. K.	No cracks
48	1¼	7	22.0	O. K.	No cracks
49	1½	3	21.0	Cracked	Cracked
50	3	..	22.0	Cracked	No cracks. Soft
51	2	10	21.0	O. K.	Soft
52	1	9	20.0	Soft	Soft
53	1¼	4	21.0	Cracked	No cracks. O. K.
54	1	4¼	23.5	Cracked	No cracks. O. K.
58	1	3¼	22.0	No cracks	Cracked
59	½	4½	21.0	O. K.	Warped
60	1½	5	21.0	Soft and crumbly	Warped and cracked
61	1	6	22.0	Warped	O. K.
62	1½	..	22.0	Did not harden	O. K.
65	1½	..	21.0	Cracked	Warped

water for 28 days. All of these cements went to pieces in cold water or in the boiling test. The results are given in Table II.

From these cements, one only, *i. e.*, No. 62, remained sound when placed in water. This cement also stood the boiling test (½ hr.), the others going to pieces. The molecular ratio of SiO_2 to Al_2O_3 for this cement is four and since the molecular ratio for good cements is between 5.1 and 6.8 and since none of these cements lie between these limits, it was decided to construct a new group. Cement No. 62 approached these ratios nearer than any other.

A new batch was calculated after Bleininger's formula ($2.8CaO,SiO_2$) + ($2CaO, Al_2O_3$) having different amounts of Fe_2O_3 and Al_2O_3 and also the ratio of SiO_2 to $Al_2O_3 + Fe_2O_3$ varied from just above to just below the limits. The using of chemically pure raw materials in place of slag and limestone gives less efficient mixtures of lime and SiO_2. It was, therefore, thought that sufficient lime would be obtained by the use of Bleininger's formula. For formulæ see Table III.

TABLE III.—FORMULÆ FOR CEMENTS MADE.

No.	Formulæ.
A_1	$5.1(2.8CaO,SiO_2) + (2Ca\ ,FeO_3)$
A_2	$5.8(2.8CaO,SiO_2) + (2CaO,Fe_2O_3)$
A_3	$6.4(2.8CaO,SiO_2) + (2CaO,Fe_2O_3)$
A_4	$7.0(2.8CaO,SiO_2) + (2CaO,Fe_2O_3)$
B_1	$5.25(2.8CaO,SiO_2) + 0.175(2CaO,Al_2O_3) + .825(2CaO,Fe_2O_3)$
B_2	$6.00(2.8CaO,SiO_2) + .175(2CaO,Al_2O_3) + .825(2CaO,Fe_2O_3)$
B_3	$6.40(2.8CaO,SiO_2) + .200(2CaO,Al_2O_3) + .800(2CaO,Fe_2O_3)$
B_4	$7.22(2.8CaO,SiO_2) + .175(2CaO,Al_2O_3) + .825(2CaO,Fe_2O_3)$
C_1	$5.44(2.8CaO,SiO_2) + .360(2CaO,Al_2O_3) + .640(2CaO,Fe_2O_3)$
C_2	$5.80(2.8CaO,SiO_2) + .400(2CaO,Al_2O_3(+ .600(2BaO,Fe_2O_3)$
C_3	$6.40(2.8CaO,SiO_2) + .400(2CaO,Al_2O_3) + .600(2CaO,Fe_2O_3)$
C_4	$7.00(2.8CaO,SiO_2) + .400(2CaO,Al_2O_3) + .600(2CaO,Fe_2O_3)$

PERCENTAGE COMPOSITION.

No.	CaO	Al_2O_3	Fe_2O_3	SiO_2	Molecular Ratio $R_2O_3:SiO_2$
A_1	66.0	0.0	11.6	22.4	5.1
A_2	66.7	0.0	10.4	22.9	5.8
A_3	67.2	0.0	9.6	23.2	6.4
A_4	67.5	0.0	8.9	23.6	7.0
B_1	66.7	1.3	9.4	22.6	5.25
B_2	67.4	1.1	8.4	23.1	6.00
B_3	67.5	1.3	7.8	23.4	6.40
B_4	68.1	0.9	7.2	23.8	7.22
C_1	67.4	2.5	7.2	22.9	5.44
C_2	68.0	2.7	6,0	23.3	5.80
C_3	68.2	2.5	5.8	23.5	6.40
C_4	68.5	2.3	5.4	23.8	7.00

These cements were prepared in the same manner except that the temperature of clinkering was determined as near as possible by the method used. The kiln was allowed to cool to about 1000 deg. C. before a batch of cement was put in and temperature was then gradually raised till clinker was formed, the temperature was then read with a Wanner pyrometer.

The clinkers obtained appeared exceptionally good, being dull black in color and glistening brightly in the sun. These

clinkers were pulverized the same as has been previously described, then tested.

The results of these tests, Table IV, show that good cements can be obtained with a large amount of alumina using the same ratio of SiO_2 to R_2O_3 as Portland cements require. One very noticeable fact, however, is that when no Al_2O_3 is present as in series A, A_2, A_3, and A_4 these cements all show expansion, thus giving evidence of free lime. Although A_1 stood the boiling test, the cubes made from this cement bulged out from the mold considerably.

The question arises at this point, is it always necessary for Al_2O_3 to be present or can a good cement be made without it?

TABLE IV.—RESULTS OF TEST.

No.	Temperature when Clinkered, deg. C.	Time to Clinker, hours.	Appearance of Clinker.	Initial Set, hours.	Final Set, hours.	H_2O, per cent.
A_1	1300	3/4	......	24	62	24.8
A_2	1320	1/2	All	22	56	24.0
A_3	1320	1 1/2	clinkered	26	56	23.2
A_4	1330	1/2	good,	28	60	26.0
B_1	1390	1/2	colored black	4 3/4	40	26.3
B_2	1320	1 1/4	and	4 1/2	44	24.4
B_3	1350	3/4	glistening	11	36	28.0
B_4	1400	1 1/2	with	5	48	25.0
C_1	1320	1 1/2	crystals	5	30	24.4
C_2	1320	3/4	in a	12	40	24.0
C_3	1330	1 3/4	bright	12	48	28.0
C_4	1380	1/2	light	17	40	27.2

This ought to be possible by reducing the lime content, as A_1 was the best of series A and also had the smallest amount of lime silicate.

The slowness of setting is another factor which must be considered. It will be seen by Table IV that all of the cements required a long time to harden. This must be carried on in a moist atmosphere also or the cement will dry out before it has completely hydrated and set. The above factors will perhaps limit the use of this cement to work under water which may be allowed to set a considerable time.

All the cements of series B stood a 6-hr. boiling test without showing any signs of expansion. In series C all but C_1 stood the boiling test. C_1 warped a little and came loose from the glass

plate although the cement has a comparatively low lime content and its formula lies between other good cements.

The attempt was next made to give these cements a comparative test with Portland cement to show their relative resistance to sea water. The method used was similar to that of Dr Michaelis.

One-inch cubes were made of each series of cements together

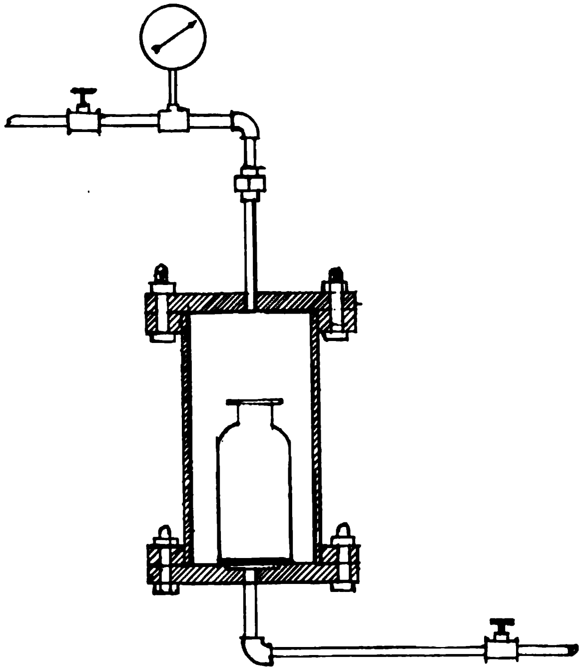

FIG. 3.—STEAM CYLINDER.

with a set of cubes of a standard commercial Portland cement, which had stood all the commercial tests. These were allowed to stand 60 hr. in the moist chamber and then placed in water, remaining in water for 27 days. The cubes made from series *A* together with a set of 5 Portland cement cubes were placed in a steam cylinder, Fig. 3, containing an artificial sea water solution of ten times normal strength. The quantity of salt is shown in Table V. The cements were then put under steam pressure

of 125 lb. or 8⅓ atmospheres, the temperature being between 150 and 200 deg. C. This was continued for 3 days. On opening the cylinder, the salt solution was found to be very dilute due to condensation of steam and no visible action on the cements had occurred. The salt solution and cubes were then put into a large wide-mouthed bottle, provided with a stopper and small vent hole. The bottle was then placed inside the pressure cylinder and steam admitted, allowing little or no condensation. After being sure that the bottle was not broken by the first change in temperature, the pressure was kept on for 3 days longer. Upon opening the cylinder, the cubes were found bone dry and covered with salt and the bottle cracked. This was due, no doubt, to the rapid reduction of the pressure, allowing the water

TABLE V.—ANALYSIS OF SEA WATER.*

Salt.	Per cent of Salt.	Ten times per cent of Salt.	Total for 12 liters of Water.
$NaCl$	77.75	.....	342.10
$MgCl_2$	10.87	108.7	478.28
$MgSO_4$	4.73	47.3	208.12
$CaSO_4$	3.60	36.0	158.40
K_2SO_4	2.46	.....	10.80
$MgBr$	0.217	.....	0.93
$CaHCO_3$	0.345	.....	1.62

37.3 parts per thousand parts water.
100 parts = 2700 parts water.

$$\frac{12000}{2700} = 4.4 \text{ factor times per cent of salt} = \text{quantity per 12 liters of water.}$$

to vaporize rapidly, which was at a temperature above its boiling point.

The results of this test were contrary to what was expected as the Portland cements were untouched and all of the iron cements were cracked and swollen. This cracking and swelling is caused, no doubt, by an excess of free lime, as these cements showed an expansion in the boiling test and there was a deposit of hydrated lime in the bottom of the cylinder which seemed to have been leached out of the cubes.

No crushing strength test of Series *A* was made as they were all destroyed already.

Series *B* was then placed in the cylinder, with a set of Port-

* *University Geological Survey of Kansas*, Vol. 7, p. 27.

land cement cubes. A vessel made of 4-in. pipe was used in place of the glass bottle to overcome cracking due to sudden change in temperature. This series was kept under pressure for 6 days, and when removed from the cylinder neither the Portland or Iron Ore cements appeared harmed except cement B_3 which went to pieces. The reason for the disintegration of this cement is unexplainable except that it was not clinkered properly. The boiling test, however, showed a good cement. (Table VI.)

As the crushing strength tests of the Portlands show, there seemed to be no weakening due to being in the salt solution.

TABLE VI.—RESULTS OF BOILING TEST FOR 6 HOURS, AFTER 60 HOURS IN MOIST CHAMBER.

Number.		Appearance after Sea Water Test.
A_1	Good.	Cracked.
A_2	cracked plate.	"
A_3	Came loose from plate and showed some expansion.	"
A_4	Same as A_3.	"
B_1	Good.	Sound.
B_2	"	"
B_3	"	Went to pieces.
B_4	"	Sound.
C_1	Came loose from plate, warped.	Cracked and swollen.
C_2	Good.	Sound.
C_3	Good.	"
C_4	"	"

Also the strength of the Portlands seems to average higher than the Iron Ore cements. (Table VII.)

Five cubes of each cement of Series C were then placed into the cylinder with a set of Portland cubes made at the same time. These were kept under pressure for 8 days. The results of this series were quite different as 4 of the 5 cement cubes were badly cracked and had begun to swell. C_2, C_3, and C_4 showed no signs of disintegration, but C_1 was cracked and swollen badly. This cement, as the A Series, did not stand the boiling test and such an action would be expected from it under the extreme conditions in the pressure cylinder. The crushing strengths of C_2, C_3, and C_4 averaged lower than the B Series, C_2 was so soft that disintegration had evidently set in.

TABLE VII.—CRUSHING STRENGTH OF CEMENTS.

No.	Cross-sectional Area, sq. in.	Crushing Strength. Total lb.	Crushing Strength. Lb. per sq. in.	Average, lb. per sq. in.
		P_1 = Portlands in fresh water 3 weeks.		
P_1	1.08	7680	7100	
	0.975	4780	4900	
	1.06	6650	6280	
	1.045	5650	4910	
	1.105	7750	7020	6042
		P_2 = Portland cement in fresh water 4 weeks.		
P_2	0.97	7850	8700	
	0.95	6620	6970	
	0.97	7730	7960	7876
		P = pressure with Series B of the Iron Ore Cements.		
P	0.97	5420	5590	
	1.25	4860	3890*	
	1.025	7650	7470	
	0.98	7330	7470	
	1.01	7200	7150	6920
		Iron Ore Cement in salt solution under pressure cylinder 6 days.		
B_1	1.035	5810	5620	
	1.075	6720	6250	
	1.035	5120	4915	
	1.06	4740	4460	
	1.045	5200	4860	5241
B_2	1.105	7170	6500	
	1.02	6620	6000	
	1.055	7500	7100	
	1.115	8430	7550	
	1.125	6680	5930	6616
B	1.09	4480	4120	
	1.075	5180	4820	
	1.10	5000	4540	
	1.06	6610	6240	
	1.12	6000	5350	5014
C_2	1.025	4200	4080	
	1.03	5400	4360	
	1.025	6320	6660	
	1.1	5850	5310	
	1.04	4850	4660	4914
C_3	1.05	2280	2190	
	0.97	1580	1660	
	1.1	2640	2400	
	1.00	1820	1880	
	1.01	2500	2480	2110
C_4	1.07	5220	3000	
	1.07	6630	6150*	
	1.06	3630	3330	
	1.07	5140	4800	
	1.04	4050	3900	5757
		Portlands in Cylinder 7 days with Series C.		
P	0.99	3000	3030	
P	0.97	6720	6930* Only unaffected Portland cement cube.	

* Signifies not calculated in average.

CONCLUSIONS.

As the time for this investigation was limited, further work could not be done, and the conclusions which may be drawn from these results are limited. This much may be said, however:

1. The amount of lime or silicate of lime ought to be less when Fe_2O_3 alone is used in place of Al_2O_3, as the lowest ratio of Series *A* 5.1 was the only one which stood the boiling test. Series *B* showed that the limits gave good cements throughout, neglecting B_3 which must have disintegrated due to some other cause. Series *C* showed that the lime and silica required increased as the lower ratio 5.44 disintegrated and the higher ratios were good. To sum this up, when all iron is used the $R_2O_3 : SiO_2$ ratio should be below 5.1; when 0.175 to 0.2 mols. Al_2O_3 is used with 0.825 to 0.8 mols. of Fe_2O_3 the ratios lie between 5.1 to 7.22. If 0.36 to 0.4 mols. of Al_2O_3 the ratio must be 5.8 or greater. This is but a suggestion and will require further experimenting to show it definitely.

2. That cements with large amounts of Fe_2O_3 will stand saline solutions better than cements containing Al_2O_3 was shown in the test of Series *C* where the Portlands were actually disintegrated and the iron cements stood the same test.

3. The results seem to suggest that if the amount of lime was reduced lower than 2.8 CaO in Bleininger's formula, better strength could be obtained. There was found in the bottom of the vessel, after each trial in the cylinder, a heavy muddy deposit which was principally hydrated lime and which appeared to have been leached from the cubes. This reduction of the amount of lime may not need to be as much as the results suggest if the raw materials were clay and limestone in place of pure whiting, $Al_2(OH)_6$ and flint. All of the iron cements would have stood the tests better if they had been allowed to stand in the atmosphere and age, thus giving the lime time to become calcium carbonate. The Portland cement, which these cements were tested against, was one of the best cements on the market. It tested as follows: Initial set, 3 hr.; final set 4½ hr.; tensile strength of neat cement after seven days, 679 lb.; after 28 days, 774 lb.; and its crushing strength is shown in the tables. This cement had also aged several months in the laboratory and was in the best of condition

to stand accelerated tests. The percent of lime given by Mr. William Michaelis is 63.5 per cent with a small amount of magnesia, MgO, 1.5 per cent. The cements made for this thesis are all above 66 per cent, this is only another evidence that these conclusions are correct and the following formula is suggested as the center of a series of cements for further experimenting:

$$4(2.8\ CaO,SiO_2)\ 0.8\ (2\ CaO,\ Fe_2O_3)\ 0.2\ (2\ CaO,\ Al_2O_3).$$

from this vary both the amount of SiO_2 and CaO.

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DISCUSSION.*

Mr. Wig.

MR. R. J. WIG.—I think it is rather dangerous to do too much theorizing on a few briquette tests. We made several thousand briquettes and from our results we can vary the curve of the results obtained by a very slight difference in the temperature of the laboratory for the first few hours of the first day, when the cement gets its original set. I think that is pointed out in Mr. Bates' paper, where he shows the effect of permitting the cement to set for several days prior to immersing in sea water, as compared with one which is only permitted to set for twenty-four hours, the one failing inside of a few weeks while the other does not fail in a year.

Mr. Bates.

MR. P. H. BATES.—The matter of most interest in Mr. Brown's paper is the fact that some one else, besides the Bureau of Standards, is using a small rotary kiln in the investigation of Portland cement. It is possibly not generally known that the Bureau established during the last year at its Pittsburgh branch a kiln of the same size as Mr. Brown's. Such an installation was both friendly and otherwise criticized. In the latter respect largely because it was claimed that it was not possible to secure results in such a small kiln that could be compared with those obtained in a large one. Up to the present we have not burned sufficient cement, nor have our tests extended over a sufficiently long period to decide this latter question.

The reason for the installation of such a kiln was just such results as Mr. Brown gives in his paper and Mr. Williams also gives in his. For instance, the former shows that certain normal Portland cements gave perfect results, also some of the high iron Portland cements gave good results, whereas certain Portland cements from the middle West and from the Lehigh district behaved poorly. There is not sufficient difference in the chemical analyses of the good and poor cements to account for this. We feel, therefore, that there must be something in the burning temperature, in the time in the kiln, in the grinding and prepara-

* Joint discussion of the preceding papers by P. H. Bates, Herman E. Brown and A. E. Williams.

Mr. Bates. tion of the raw material, etc., which influences the results to a decided degree.

Mr. Brown refers to having made dicalcium aluminate. This material does not exist. The aluminates which may exist in Portland cement are $3\ CaO.\ Al_2O_3$; $5CaO.\ 3Al_2O_3$ and $CaO.Al_2O_3$.

In regard to the difficulty encountered in the burning of iron oxide cement, as mentioned by Mr. Brown, it might be stated that this has been noted before. The vitrification range, as ceramists would call it, for such raw mixes is very narrow, and further the tendency to produce rings in the kiln is also very great.

The author also calls attention to the increase of heat, or rather the development of heat noticed in the clinkering zone, caused by the reaction between the lime and silica. We have also noticed this in our small kiln; and, while we acknowledge that there is a considerable heat of reaction between the elements in question, we have not considered that the very great heat of the clinkering zone is due entirely to this heat of reaction. It should be remembered that combustion is more perfect at this point and further there is usually a reduction of the area of the kiln due to rings and clinker adhering to the walls of the kiln.

The following paragraph is somewhat indefinite:

> When the raw materials are ground to the same degree of fineness, it is not possible to carry the lime to the silicates ratio as high in Marine cements as in aluminous cements, if sound cement is to be secured. This behavior indicates that ferric oxide is not a complete substitute for the alumina, and cannot carry relatively as much of it in combination, molecule for molecule, as alumina.

On account of the greater molecular weight of iron oxide, it follows directly that the percentage of lime combined with iron to form a definite compound will be less than that combined with the alumina to form the same compound. It follows directly, therefore, that the lime silica ratio of a cement containing a definite amount of alumina would have to be greater than one in which the alumina is replaced by the same amount of iron oxide.

There are too many variables in the cements under investigation by the author to conclude that cements low in magnesia are required for sea water use. In all cases which the author gives these variables are of as much importance as the magnesia, and to conclude that low magnesia is desirable is not warranted.

It is interesting to note that Mr. Williams found it possible to secure Portland cement of ordinary composition which behaved in sea water as well as iron ore cements, *i. e.*, examining his results, it is found that he made three burnings of iron ore cement and compared these with the Portland cement. In the first case the Portland cement withstood the sea water action very well, while the iron ore cement went to pieces. In the second case, the behavior of the two was about equal. In the third case the Portland cement went to pieces, while the iron ore behaved the better. From these results alone it is evident that the Portland cements behaved as well as the iron ore cements. **Mr. Bates.**

FLAT SLAB CONCRETE BRIDGES.

By William H. Finley.*

Since the introduction of concrete for the construction of railroad bridges no type of structure has met with more success than the flat slab bridge. In the days of stone masonry work was confined to the flat top culvert for openings up to 4 or 5-ft. spans; beyond that it was necessary to use the arch type, which meant increased expense in construction due to the cost of the cutting of the masonry for the arches. For years after the introduction of concrete it was a common practice to use, for small openings say up to 8 or 9 ft., old railroad rails embedded in concrete to form the top covering. In those days there was more or less second-hand rail that could be used economically for this purpose, and it was thought that in using the material in this way a cheap and satisfactory structure could be produced. It was soon discovered, however, that second-hand rails were not always available and if available they could be put to better use than burying them in concrete for culvert construction.

In spans greater than 9 ft. and up to 30 ft. it was the practice, for a number of years, to use I-beams embedded in concrete. Among some engineers the idea was to use beams of sufficient strength to carry the live load and consider the concrete only as taking the place of lateral bracing. This form of construction required very careful work in the field, inasmuch as it is difficult to effect a satisfactory bond between concrete and large areas of metal, as represented by an I-beam, and with the greatest care in the selection of the material and the placing of the concrete it was difficult to prevent the same from cracking or separating from the beams. Although there were a number of satisfactory bridges built of this type, yet it was gradually abandoned.

As confidence grew in reinforced concrete these smaller openings were built of the reinforced flat slab type (Figs. 1, 2 and 3). This type not only took the place of stone arches of corresponding span but also greatly reduced the cost. Engineers were

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enabled to satisfactorily solve a great many problems in places where a wide and low opening was particularly desirable. The ideal structure from a railroad point of view is that type which entails the least cost in maintenance. Stone masonry, however carefully constructed, required from time to time repointing to prevent the infiltration of water and consequent disintegration of the structure. With concrete properly constructed this maintenance charge is completely eliminated. Not only is the main-

FIG. 1.—A FLAT SLAB CONCRETE CULVERT OF 10-FT. SPAN.

tenance charge eliminated but the first cost of construction is considerably reduced. In the building of a flat slab bridge as a monolith it is possible to use less material in the side walls than would be required for an arch of corresponding span.

In railroad construction numerous instances occur where it it is necessary to provide a wide and low opening and this condition is admirably met by the flat slab type of bridge. Before the introduction of this type of construction it was necessary to provide an arch that would entail very much more expense.

Of recent years the flat slab bridge has been used to good advantage in cities where the railroads were eliminating grade crossings by the elevation of their tracks. In the ordinary 66-ft. street where the city permits supports in the center of the street and on the curb lines it is possible to use the flat slab type of bridge for spanning these openings. This results in a very economical form of construction as well as one that can be made aesthetically pleas-

FIG. 2.—A FLAT SLAB CONCRETE BRIDGE OF 16-FT. SPAN.

ing in general appearance. It also reduces to a minimum any maintenance expense. A steel structure for a similar opening requires painting every 4 or 5 years and if this work is not attended to the structure soon presents an unsightly appearance. This condition is very prominent in our elevated railroads in cities where the maintenance is neglected.

Twenty years ago, in railroad construction, it was the universal practice to use, in openings from 6 to 30 ft., open deck I-beams. These were usually placed upon concrete or stone ma-

sonry abutments and presented as much difficulty in maintaining good track as much larger spans for the reason that the dump ties were just as troublesome to keep up as the same ties on a 150-ft. span. There is hardly no greater use for flat slab bridges than in the replacing the open floor I-beam spans. It is usually possible to remove the I-beams and replace them with flat slab bridges, carrying the ballast across the bridge and making a uniform track surface. The I-beams can be used for other purposes or they have

FIG. 3.—A FLAT SLAB CONCRETE BRIDGE OF 20-FT. SPAN.

at least a salvage value that goes toward reducing the cost of the improvement. A certain stretch of railroad of about 36 miles has over 20 small openings from 6 to 25 ft. made up of I-beams with open floors. However carefully the track work was kept up it meant a bad riding track as each one of the bridges represented an unyielding space that was very noticeable to the traveling public.

There is no class of reinforced concrete construction, however, that is entitled to more careful and intelligent supervision in the

construction of the same than flat slab bridges. When concrete began to displace stone masonry unfortunately the sole idea of constructing engineers was to see how cheaply they could build structures in concrete. Apparently many engineers were so anxious to prove the superiority of concrete in both cost and durability that they frequently sacrificed the latter for the former. It is generally recognized today that to produce satisfactory concrete construction it is necessary to use the utmost intelligence and care in the selection of the materials and in the execution of the work. Unfortunately reinforced concrete construction has, in the past, been exploited from the commercial side rather than from the engineering. This, I think, was largely due to the various patented types of reinforcing material. Concerns marketing reinforcing material looked upon it from the commercial side only and furnished all sorts of information as to not only the good points of their particular shape or type of reinforcement but also formulas and information as to the construction of reinforced concrete that would apparently enable any one, engineer or otherwise, to undertake the work of designing and building reinforced concrete structures. Happily we are passing out of that stage of development and it is now being generally recognized that it is necessary to have engineers of experience and training to correctly design any reinforced concrete structure.

CONCRETE HIGHWAY BRIDGES.

By Walter Scott Gearhart.*

The pictures of good and bad bridges herein presented are not expected to add anything new to the scientific knowledge of bridge design and construction, nor were they collected for that purpose. In dealing with county, township and city officials, I have been convinced that it takes more than ordinary argument and plain statement of facts, which they often cannot comprehend, to convince them that there are better ways and means of doing things than those used by their "dads." In Kansas our officials all seem to be Missourians, for they have to be shown. I have been using the pictures very effectively by showing poor, damaged, and wrecked bridges in contrast with first-class, permanent structures and comparing the life and first cost and the maintenance expense on the different kinds of bridges and culverts. This I find to be most convincing, especially when accompanied by a statement of the taxes used each year in the particular county for bridges and culverts.

There is an abundance of excellent text books on bridges and culverts and their construction and design, and there seems to be ample space given on the program to the technical phases of this subject. It is not more engineers we need, but public officials educated to use the good ones we already have and to impress upon the public the economy of employing competent engineers to supervise the construction of their bridges.

The pictures were selected for their educational value to the layman who has given but little thought to this problem, and I am satisfied these are the most convincing arguments one can use. If the pictures or anything I might say will help any delegate or suggest any means to him by which he can convince his people that it is economy to build better bridges and culverts, this paper will have served its purpose.

There is little question but that at least the small bridges and culverts can be made the most permanent places in the public

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highways. In Kansas, Nebraska, and Oklahoma at the present time the bridge problem is very much more important than the surfacing the roads, because to use the roads the bridges and culverts must be kept up, and they cost just as much one place as the other for they will rot, rust and wash out just as quickly in the lane as on the heavily traveled road.

At the worst and the most dangerous places on the highways are located the light wood and steel bridges and culverts and in most cases these are the places where more money has been spent than anywhere else on the road. Surfacing the roads with brick as is being done in a number of eastern states at a cost of from $10,000 to $20,000 per mile, they would then probably not last more than 25 to 30 years at the outside. In contrast with the brick roads, it is possible to build bridges and culverts of materials which make them practically everlasting and almost entirely eliminate the maintenance cost. Kansas is spending annually $5,000,000 for roads and bridges, and $2,500,000 of this is used for bridges and culverts. More than 40 per cent of this is used for repairs to old bridges and culverts and in addition to this there is a great waste due to ignorance, poor materials, bad designs, politics, and loose methods of handling contract work.

Many of the good and bad bridges and culverts illustrated we are not very proud of and do not care to advertise, but in order to arouse public sentiment a sufficient amount to get action it seems necessary to have an Iroquois fire, a Cleveland school, or a New York shirt waist factory holocaust. It is almost necessary at times to dynamite the public before any considerable number of persons will give up their old customs and fixed ways of doing things.

A culvert composed of two 18-in. galvanized corrugated ingot iron pipes with wood head walls is shown in Fig. 1. The total cost of the culvert was about $55. If one 24-in. pipe had been used, it would have carried as much water and would have cost $15 less. If a reinforced concrete box 30 x 18 in. had been used, it would have carried more water and proper concrete head walls could have been put on, thus making the structure permanent at a total cost of not exceeding $50. This township board, like almost all the others, needs the advice of a competent county or district highway engineer.

FIG. 1.—CULVERT OF TWO 18-IN. CORRUGATED PIPES WITH WOODEN HEADWALLS.

FIG. 2.—A TYPICAL WOODEN STRUCTURE.

A good illustration of the waste of public funds by building bridges larger than necessary is shown in Fig. 2. This structure is 44 ft. long and 16 ft. high and drains 160 acres. A concrete box 5 x 5 ft. would carry all the water that would ever have to pass through. There are hundreds of such places in the state and just as many more where the culverts are as much too small as this one is too large.

It is the kind of a bridge one would gladly drive around if possible, and often have to do in order to reach the opposite side of the stream. It is little less than criminal to maintain such

FIG. 3.—THE DANGER OF FLIMSY STRUCTURES.

structures on a public highway and a disgrace to any community and a menace to the public. In most cases these structures are at the worst and most dangerous, the most expensive, and the most inexcusable, indefensible places in the road, and enough money has been spent on many of them to have made them permanent structures long ago.

An old wood pile bridge near Lincoln, Kan., collapsed after a drove of steers had been driven on it, killing nine fat steers. A short time after this failure, another such bridge near Burlington, Kan., collapsed just after a surrey loaded with people had crossed it. Many of these old wood and light steel structures stand only by force of habit.

Fig. 3 shows a bridge near Manhattan, Kan., with engine in the river. The engine driver was crushed and drowned under the engine when it fell. The front wheels were 5 ft. out on the steel span when the bridge failed. The ends of the stringers were badly dapped, or cut down, and the weight of the rear wheels of the engine simply split them off.

The effect of a flood on a bridge near Russell, Kan., is shown in Fig. 4. The board depended entirely upon a bridge agent for their engineering advice. The structure is composed of two 80-ft. spans and six 26-ft. spans. The superstructures are heavily overloaded with the concrete floor and the foundations were built

FIG. 4.—A RESULT OF "FREE" ENGINEERING ADVICE.

by driving 10-in. I-beams 32 ft. long into the sand,—not to refusal, but until they were down enough so that the floor could be placed at the elevation desired. The I-beams were then encased in concrete from the bed of the stream up. The bridge sets on an angle with the stream and when the flood came the water scoured around the foundations and the short spans sunk about 4 ft. as shown in the picture. Since this picture was taken, the county board had the bridge jacked up and tried to strengthen the foundations, but I understand it has sunk again.

Fig. 5 is a view of one of the first reinforced concrete bridges built in Wabaunsee County. It has a 20-ft. span, 16 ft. roadway and contains 80.1 cu. yd. of concrete and 3,800 lb. of steel rein-

forcement. ·Materials were hauled 12 miles and the cost of the structure complete was $895. The Office of the State Engineer prepared the plans. Note the pleasing appearance the paneling gives the railing.

Another reinforced concrete girder bridge with a 25-ft. clear span and a 16-ft. roadway, supported on stone abutments, cost complete $712. The plans were prepared by the Office of the State Engineer. The substructure contains 55 cu. yd. of stone masonry and the superstructure, 24.9 cu. yd. of concrete and 4,719 lb. of steel reinforcement.

FIG. 5.—REINFORCED CONCRETE BRIDGE OF 20-FT. SPAN AND 16-FT. ROADWAY.

Fig. 6 illustrates a 30-ft. reinforced concrete girder bridge and clearly shows the advantage of this type of construction over an arch in the flat country. It gives the same waterway as a wood or steel truss bridge of the same span. If an arch had been used, it would have been necessary to make the span 25 to 30 per cent longer, or very much higher to carry the same amount of water. A hump in the road, caused by increasing the height of an arch, is very objectionable and often the increased length of span would not be desirable. In most localities in the central west, the bridge

foundations rest upon clay, loam, or sand and it is expensive to carry the footings down to great depths or to drive piles to make the foundations so solid that no slight settlements will occur, which is absolutely necessary in constructing concrete arches.

In the case of the reinforced concrete slab or girder bridges, there is little danger from this source, for all of the forces are vertical and if considerable settlement should take place, it would not injure the superstructure. I know of a case where one abutment under a 30-ft. span settled 11 in. without injuring the top in the least. The top was afterwards jacked up and the abutment underpinned and built up to the proper height.

FIG. 6.—REINFORCED CONCRETE GIRDER BRIDGE OF 30-FT. SPAN.

The cost of reinforced concrete slab and girder bridges is generally considerably less than for arches up to 50 or 60-ft. spans, and often multiple girder spans can be used more economically than arches for structures over 50 ft. long.

The above is not intended as a criticism of the arch type of construction, but is given simply to point out the advantages of the slab and girder bridges. Both types of construction are equally strong when properly designed, but many laymen are skeptical about reinforced concrete slab and girder bridges.

A view of two reinforced concrete girder spans each 35 ft. in the clear, with a 16-ft. roadway, between Butler and Harvey

counties, is shown in Fig. 7. The plans were prepared by the Office of the State Engineer, and Charles Buskirk, County Engineer, superintended the construction. The foundations contain 241.4 cu. yd. of concrete and the superstructure, 88 cu. yd. of concrete and 14,962 lb. of steel reinforcement. The contract price of the bridge was $3,100.

Plans for a heavy steel bridge with a concrete floor and concrete abutments were prepared and the lowest proposal made for such a structure was $3,800. However, the structure was not

FIG. 7.—REINFORCED CONCRETE GIRDER BRIDGE OF TWO 35-FT. SPANS.

worth over about $3,450. Prices were also received at the same time for a light steel bridge with a wood floor and the lowest price received was $3,300. The commissioners rejected all bids and adopted the concrete plans and, as stated above, the bridge was completed for $200 less than the cost of the lightest kind of a steel bridge which would have required constant repair. These counties are not building any more light steel bridges, but are putting in concrete structures wherever it is possible.

Fig. 8 shows the Melan arch at Topeka, Kan., over the Kaw River. This was one of the first large reinforced concrete arches

built in the United States. It has successfully withstood a number of exceptionally high floods when most of the steel bridges above and below it were washed away.

Through the courtesy of Mr. R. E. McDonnell of the firm of Burns and McDonnell, Consulting Engineers, Kansas City, Mo., who kindly furnished me the following pictures, I am able to present some interesting stone and concrete masonry work done at an early date.

Fig. 9 is a view of one of the old Roman aqueducts and close examination will show that the interior lining of the aqueduct is of

FIG. 8.—MELAN ARCH REINFORCED CONCRETE BRIDGE AT TOPEKA, KAN.

cement. The different grades of masonry used are due to the fact that the aqueduct was destroyed in different wars and was reconstructed at various periods. The periods were often 100 to 150 years apart and at each different period a different class of masonry was in practice.

Fig. 10 is a picture of an old Roman sewer built during the the reign of Cæsar, and Mr. McDonnell states that he chipped out some of the cement used in the masonry work and upon chemical analysis found it to be exactly similar to our present Portland cement. This old Roman sewer is now 80 ft. below the street

level. It was built large enough for navigation purposes, and a portion of the river Tiber is diverted through the sewer for cleansing purposes.

After viewing the remains of the dangerous, temporary expensive work of the Kansas bridge builders from the beginning

FIG. 9.—OLD ROMAN AQUEDUCT.
FIG. 10.—OLD ROMAN SEWER OUTLET.

to the present time and the permanent work now being done and the old structures built by the Romans, which are in use today, there can certainly be no question in the mind of any rational man as to the safest, most satisfactory and the most economical kind of bridge to build in the central west.

CONCRETE BRIDGES.

By Daniel B. Luten.*

A concrete arch, in harmony with its surroundings, but without ornamentation or embellishment of any kind, is an exceedingly beautiful structure. A concrete girder under similar conditions may be an exceedingly ugly structure. Why is it that the concrete girder bridge presents so undesirable an appearance, while the arch bridge is such a handsome improvement? Not alone because of the natural beauty of the surroundings; not merely because of the white-concrete in the arch view. The concrete arch presents the more pleasing appearance because it employs concrete in a thoroughly natural manner (Fig. 1). Every one understands that concrete is a masonry material and that, like stone, it is strongest in compression, and while it is true that steel may be embedded in concrete to resist tension it is not always easy to show its presence by external design. Hence the eye is offended by the long span girder as it never is by even the flattest of arches. The layman may be surprised by the lightness and boldness of some of the arch designs but the surprise is of a pleasing kind, whereas the feeling with respect to the long span concrete girder is one of wonder that it stands at all, a warning to stand from under. Almost any kind of a concrete bridge, however, is better than the temporary wooden and steel bridges that have been erected on our highways in times past.

Concrete as a structural material is full of surprising possibilities and one of these is that the most beautiful and appropriate applications of concrete to bridges, that is in the arch form, is also the most satisfactory from almost every engineering standpoint. In steel bridges, the arch is difficult to fabricate and difficult to erect, and consequently steel trusses and girders have a distinct advantage. Steel girders moreover are better adapted to soft foundations than steel arches. But in concrete bridges the very reverse conditions apply. It is the concrete arch that is easier to erect than the concrete girder and it is the concrete arch

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FIG. 1.—CONCRETE ARCH BRIDGE, BOULDER, COLO., 70-FT. SPAN.

that is better adapted to soft foundations than the concrete girder.

The very cheapest forms of steel highway bridges have been the "bedstead" bridges (Fig. 2), in which the beams or trusses are supported by steel legs resting on wooden sills in the bed of the stream and with wood or stone backing behind the legs to support the earth fill. Such a bridge has never been considered anything but a makeshift and when the funds are available, is usually displaced by bridges with masonry abutments. The cheapest form of concrete girder bridge is a reproduction of this steel bedstead in concrete (Fig. 3). This concrete bedstead bridge still retains most of the objectionable features of the cheap steel bridge especially for spans of more than 20 ft. In fact, the only defect

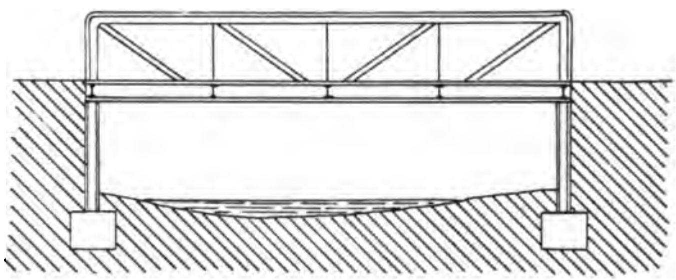

FIG. 2.—STEEL BEDSTEAD BRIDGE.

of the steel bedstead bridge that is eliminated in its concrete successor is the rusting of the legs, not a fatal defect in any event, for such bridges usually fail from other causes long before the legs rust off.

The bedstead bridge in steel is objectionable because it has no provision for expansion or contraction and no proper support for the earth fill which tends to buckle the legs together. The bedstead bridge in concrete retains these same defects. Instead of being supported on stable abutments permitting expansion and contraction of the girder, it is supported on legs, an exact counterpart of the objectionable and cheap steel bedstead. In fact if the steel bedstead bridge were encased in a shell of concrete the result would be the concrete girder in external appearance. This type of concrete girder is so inefficient in its application of

concrete that leg supports are almost essential to make it a competing structure in cost with concrete arches and even then it cannot be built at the same cost as arches except for the shortest spans, of less than 20 ft. say. Approximately one-eighth of the concrete in the girder is doing effective work in resisting compression; the other seven-eighths serve merely as a protective coating for the embedded steel. In a concrete arch the material is in much more nearly uniform compression throughout, and economy is therefore attainable with lower stresses and greater safety.

By building a concrete bedstead bridge instead of a steel bedstead, the rusting of the legs is eliminated but a much more serious defect is introduced. The support may be removed from under one of the four corners of a steel bridge and the structure still remain standing because of its floor and lateral connections.

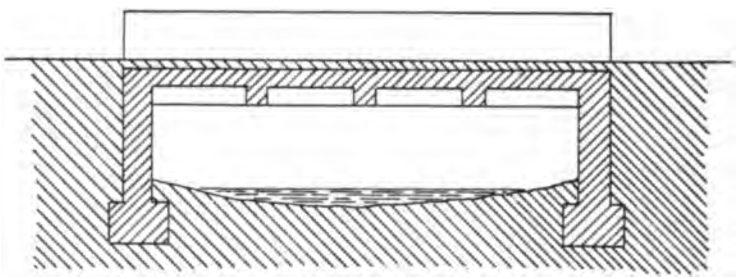

FIG. 3.—THRONG GIRDER OR BEDSTEAD CONCRETE BRIDGE.

But in the concrete girder or bedstead bridge any unequal settlement of foundations is immediately fatal. The concrete bedstead bridge has a solid concrete floor slab hung between the two girders. Settlement of an abutment is almost invariably unequal and since the floor connections cannot sustain the heavy girders the floor slab must sustain distortion when settlement occurs at one corner (Fig. 4). It must be sheared longitudinally to assume the distorted position. Such shearing will, of course, wreck the floor slab and permit overturning of the girders with complete destruction of the bridge. Any abutment settlement therefore under a concrete bedstead bridge will result in collapse unless the settlement be absolutely uniform. The concrete arch on the contrary by cracking in two lines diagonal to one another can adjust itself to any unequal settlement. Such cracks in the arch are invisible

for small settlements, are no more objectionable than the joints in stone arches and are evidenced only by cracks in the spandrel walls which may prove unsightly, but can at least be corrected if necessary by replacing the wall since it is nothing but a retaining wall for the earth fill of the roadway.

An 80 ft. arch at Pasadena, Cal., settled 8 in. at the crown, due to lack of balance of adjacent arch thrusts. The settlement continued for several months until it was finally stopped by placing a support under the crown. At South Bend, Ind., an arch of 120 ft. span settled 6 in. over night during construction and is now in use in that distorted condition with no visible evidence of its

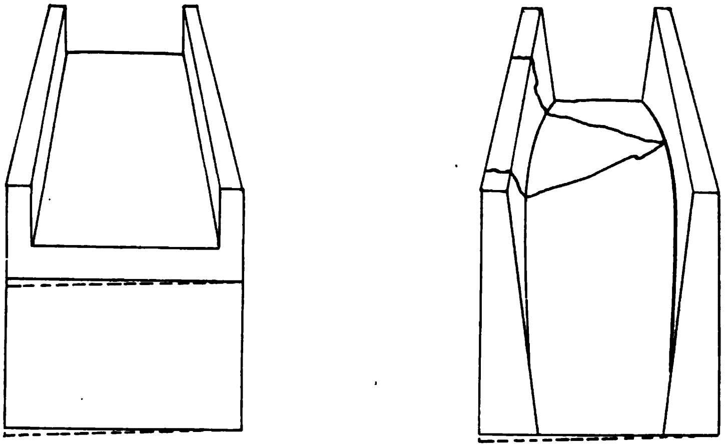

FIG. 4.—DIAGRAM SHOWING EFFECT OF UNEQUAL SETTLEMENT OF FOUNDATION ON A GIRDER AND AN ARCH BRIDGE.

misfortune. At Jacksonville, Fla., two earth filled arches of approximately 60 ft. span each, resting upon an intermediate settling pier have been jacked up 18 in. as the pier settled. Very few complete collapses of arch bridges have occurred, and those few have been due mostly to undermining floods. Whereas numerous unexplained failures of well-designed girder bridges have been reported. The evidence seems to be conclusive therefore that it is the concrete arch that is best adapted to soft foundations and can even be jacked back to place, and that it is the concrete girder bridge that ought to be built upon rock foundation rather than the arch.

It is sometimes urged that the horizontal thrust of the arch renders it objectionable for soft foundations. But this is an element that can be easily provided for by any proper study of the foundation conditions. One might with equal propriety contend that the great compression in the extreme fiber at the middle of the girder was an objectionable feature of the girder. The one can be provided for by the expert as readily as the other.

Arch failures occur slowly and with abundant warning. Girder failures occur suddenly and usually without any warning whatever. There are numerous instances of cracked arches still standing and doing good service. But the cracked girders have almost invariably been followed by collapse.

The reinforcement required for a concrete bedstead bridge is a mass of intricate detail requiring the greatest care on the part of the builder. Whereas the reinforcement for a concrete arch is simplicity itself. Since an arch contains but a small amount of reinforcement and requires that only for live load stresses, the concrete arch will increase in strength with age as the concrete hardens. The concrete girder relies for its strength mainly upon the embedded reinforcement even for the dead load stresses, and does not therefore increase in strength with age. Moreover, the compacting of the earth fill over the arch is a source of strength whereas it is an element of weakness in the bedstead bridge.

The roadway width over a concrete arch can always be widened without destruction of the original structure. In the case of the girder bridge with its roadway suspended between the two girders, the increased width can be secured only by tearing down the original superstructure. Since such a bridge cannot be built to advantage with a wider roadway than 18 ft., which will not be sufficient in many communities in fifty years, such a bridge lacks a great deal of being a permanent improvement. An arch may be built with narrow roadway and the cheapest of finish and with no ornamentation, to suit the present needs of a community since at any time the growth of a community in traffic and in value may be provided for by adding increased width to each side of the roadway of the arch, modifying at the same time the ornamental treatment to suit the growth of the community. Such additions to the arch structure will, of course, completely hide the arch as first erected.

The waterway area of an arch is always greater than that of a concrete beam or slab bridge in which the roadway is carried upon the girders (Fig. 5). But when the roadway is carried between the girders, the advantage is in favor of the girder bridge, although not as great as is generally supposed since the rectangular opening is not as effective for flood discharge as the arch opening. This will be clear from a consideration of the three possible types of waterway openings, V-shaped, rectangular and A form of equal area. The V-shaped opening is suitable for sewers in which rapid flow is desirable for carrying solids. But it is ill adapted to bridges because it restricts the waterway at low levels, thus damming it back and producing a flood and because it requires a long span. The A opening is the other extreme, susceptible of being spanned by an arch structure of shallow depth at crown and having a long span at the lowest level to carry flood waters away before

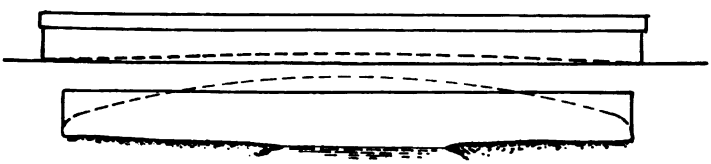

FIG. 5.—COMPARISON OF AN ARCH AND GIRDER BRIDGE IN A FLAT LOCATION, 60-FT. SPAN.

they can be dammed back. It has the lowest center of pressure and will discharge most at any flood level. It moreover provides an opening for ice or débris at a higher flood level than either of the others. It will usually be found, therefore, that while an arch may have 10 to 20 per cent less waterway than the corresponding bedstead bridge, its flood capacity is not by any means limited to the same extent. An arch is properly cambered about one-tenth of an inch to the foot of span for appearance. This camber further increases the possible waterway opening. The objections sometimes made to an arch, therefore, that it is not adapted to low, flat locations is not usually founded on good judgment. In the arch foundations, the load is supported at the back of a wide abutment, while in the concrete bedstead bridge, the load is transmitted to the foundations at the stream face of a narrow abutment. This difference gives the arch greater immunity from

undermining. In nearly all flood failures, it has been the narrow pier and not the wide abutment that was undermined.

Concrete expands and contracts with changes of temperature at approximately the same rate as steel. In this climate the range in such structures may be 80 or 100° F. For a span of 100 ft., a rise of temperature of 100° will increase the span approximately $\frac{3}{4}$ in. The bedstead bridge has no provision whatever for sliding on the abutments, the girders being securely joined to the legs

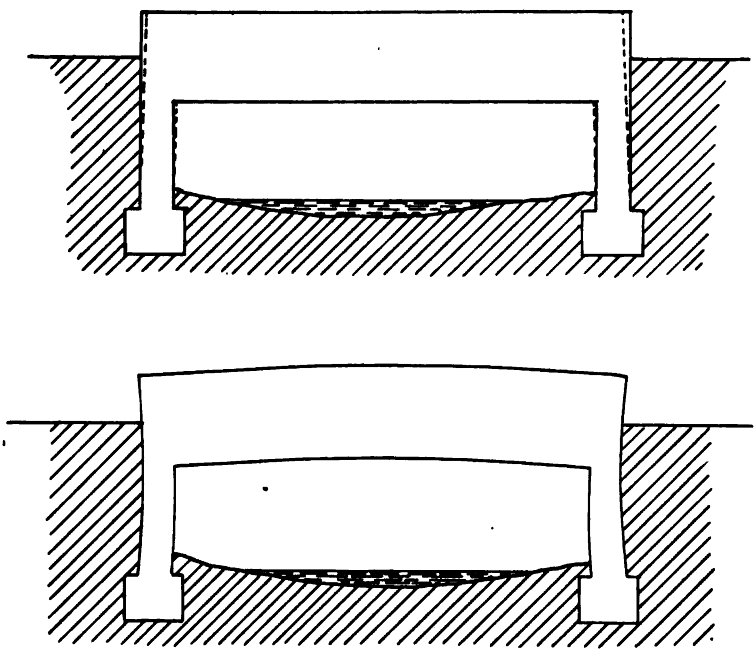

FIG. 6.—EFFECT OF TEMPERATURE CHANGES ON CONCRETE BEDSTEAD BRIDGE.

themselves. Hence in winter the tops of the legs are drawn towards each other and in summer, are forced apart (Fig. 6). This movement recurs with every change of temperature and there must, therefore, be a continual following up and pushing back of the earth fill. If the pounding of traffic does not accomplish this, the action of frost most assuredly will. This must result in stresses tending to curve the legs inward and to arch the girder upwards. Steel bridges of this type are frequently found with a lower chord

in compression throughout, the upper chord carrying the loads by arch action alone. Of course, the stresses in such a structure are utterly indeterminate.

The concrete arch provides automatically for changes in temperature (Fig. 7). The abutments remaining fixed the changes in length of the arch ring produce a slight rising or falling of the crown of the arch. It is necessary only to provide expansion joints in the spandrel walls to avoid the cracks that otherwise might occur from this arch movement. The distortion of the arch ring does, of course, modify the stresses and some designers have found such stresses excessive for the lowest temperature combined with the heaviest loading. No failure has ever occurred under these conditions, and the high stresses found have undoubtedly been due to a strict application of text-book methods of analysis

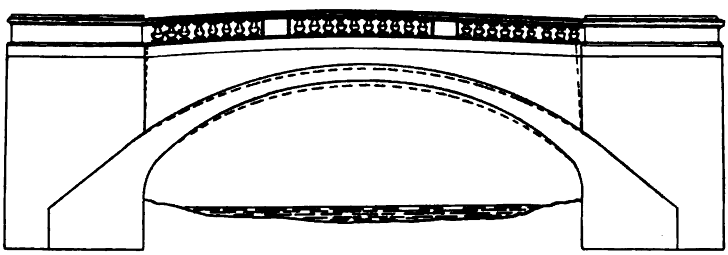

FIG. 7.—EFFECT OF TEMPERATURE CHANGES ON A CONCRETE ARCH BRIDGE.

without good judgment. For example, the excessive stresses can usually be reduced by decreasing the thickness of the arch ring at the critical point. In other words, thinning the arch will strengthen it.

Assuming the abutments to be elastic, as well as the arch ring, in the application of the elastic theory will eliminate these excessive stresses altogether. Arches carrying heavy brick buildings have behaved satisfactorily for years, proving that the arch movements due to temperature changes are not more serious than the usual settlements in foundations of such buildings. The stress in the arch ring at the point where it joins the abutment cannot be nearly as serious as the stresses on the bridge seat of the bedstead girder due to eccentric application of load with change of span caused by variations in temperature.

The pavement on a bridge should be of the same character as the paved approaches. The practice of using a light, reinforced concrete floor without a covering of earth or other paving material cannot be too severly condemned, whether for steel or concrete bridges. Such a floor is hardly permanent more than a wooden floor yet is frequently used on the assumption that it makes a permanent bridge and because of its lightness which permits reduced sections in the bridge. Any flat top bridge is objectionable under a roadway of earth or gravel because a chuck hole forms at each approach, due to the change from a harder to a softer foundation. No such depressions occur over arches.

The concrete arch thus has the advantage of the concrete bedstead bridge in almost every respect. It is better adapted to soft foundations, is more efficient and more economical, provides greater discharge for the same waterway area, is not injuriously affected by temperature changes, provides a smooth roadway, is easier to erect, and fails slowly in case of failure, can be widened at any time without loss of the original investment and last and most important of all is adapted to artistic treatment of railings and spandrels in great variety.

The arch is the ideal bridge for the expert designer; it is intricate in analysis and not readily solved by the novice. Many improvements have been made in late years to reduce its cost and many more are possible. Details of false work centers have been improved in the last ten years to save 50 per cent of their cost. Piers have been lightened to one-twentieth the span where a rigid application of the elastic theory according to text-book methods would require three times that thickness. The concrete arch has thus become the cheapest of permanent bridges for spans in excess of 20 ft. and is the ideal highway bridge.

DISCUSSION.*

Mr. Allen Brett.—While the elevation of the steel bedstead bridge, as it is called, is very much like a bedstead, I hardly think it is fair to call the concrete bridges bedstead bridges. What would be the actual lengthening in the girder of a bridge, say of a 40-ft. span, for the ordinary range of temperature? Mr. Brett.

Mr. Daniel B. Luten.—I would assume a variation of 80 to 100 degrees. The shortening due to temperature in such a bridge would be $\frac{5}{16}$ of an inch, not a serious amount at all. But nevertheless it should be provided for. Expansion rollers are used in long steel bridges because of their greater loading. A short span concrete bridge has a greater load than a long span steel bridge; and for the same reason then rollers should be used in the short span concrete bridge. Mr. Luten.

Mr. Walter Scott Gearhart.—One might suspect from Mr. Luten's remarks that reinforced concrete girder bridges were not satisfactory and that the patented arch bridges he recommends were the only ones worth building. Mr. Gearhart.

It is an easy matter to take care of the expansion on girder bridges and this point need not alarm anyone. For spans up to 40 ft., cast iron plates can be used satisfactorily and for the longer spans, cast iron rockers embedded in asphalt are just as satisfactory as rockers or rollers under a steel bridge.

If Mr. Luten will make a practical application of these fine arch adjustments on one of the patented arches near Lyndon, Kansas, which is badly cracked, it will be a relief to the County Commissioners who are responsible for its construction.

Mr. Luten.—My objection is to the use as a pavement of a structural member consisting of a reinforced concrete beam or slab, since the upper surface may become worn by traffic, and because of that surface being a compression member, failure would result from its removal in ten or twelve years. Mr. Luten.

* Joint discussion of preceding papers by William H. Finley, Walter Scott Gearhart and Daniel B. Luten.

Mr. Lindau. MR. ALFRED E. LINDAU.—I want to ask Mr. Luten if it is not possible to put some kind of gravel covering on top of the reinforced concrete slab as well as this on the arch, if it is not usually the practice to put the concrete surface and then a wearing surface on a structure of that kind to take care of this feature?

Mr. Luten. MR. LUTEN.—Yes, and that is good practice. I am criticizing the case where the structural member is used for the floor surface itself. If 12 in. of gravel were placed on top of the slab in a concrete bridge it would provide complete protection. Six inches would hardly be sufficient. A concrete wearing surface added to the slab properly designed would be satisfactory. If 3 in. of concrete be added on top of the concrete slab, with a satisfactory finish, to be replaced with another pavement when worn out, the result is satisfactory. The practice I am condemning is the use of a concrete slab as a carrying member and a wearing surface at the same time, in order to reduce the load on the bridge and thus reduce the concrete in the structure. That is a very common practice in some parts of this country.

Mr. Lindau. MR. LINDAU.—I would like to ask if it is not a fact that temperature changes and questionable foundations are not more serious in arch construction than they are in the ordinary girder construction or concrete beam construction. If the foundations of an arch settle do they not make the stresses in an arch floor more indeterminate than they would in a concrete girder—just speaking of the efficiency or desirability of the girder type from an engineering standpoint?

Mr. Luten. MR. LUTEN.—I have said all on that subject I can, except perhaps to accentuate this difference, that when a girder settles it fails. A $\frac{1}{8}$-in. settlement in a girder bridge may not cause failure, because the solid floor may be able to deflect that amount; but one inch of settlement makes it fall absolutely down, across the corner. Now a 1-in. settlement in the corner of an arch bridge does not mean failure at all. I have seen them with far more settlement than that. Foundations go down without any settlement at all. The argument I make is that the concrete arch is better adapted than the concrete girder to soft foundations, whereas some engineers recommend the arch only for rock foundations, and they are mistaken.

A Member. A MEMBER.—I would like to inquire as to the comparative

utility and advantages between the type of flat slab bridges just referred to, the true girder bridge, as against one with the girders placed below, say four girders below the floor, in the case where the height of waterway is not in question and there is plenty of water. A Member.

MR. GEARHART.—In most cases, I believe the T-girders would be cheaper. The only object in putting the girders on the sides is to increase the waterway, which in many cases is a very important matter, but this type of construction requires more concrete and I believe it would require more reinforcement than for the T-beams. Mr. Gearhart.

I would like to ask if anyone has experimented with any other materials except gravel or macadam for the surface of concrete floors on steel bridges? I have in mind asphalt mixtures or compounds or something of that nature which would give a light, durable wearing surface.

MR. WILLIS WHITED.—In Pennsylvania we recommend, whenever the pavement is made of brick, to merely put a sand cushion on top of the slab on the bridge and then the paving on top of that, and, similarly, with other kinds of paving, the idea being to let the slab of the bridge floor serve the purpose of the concrete foundation, roughing it up, if necessary, to prevent the pavement from creeping. Mr. Whited.

I have never had any experience with a thin pavement, but, where asphalt pavement is used, the ordinary asphalt binder could be laid directly on the concrete slab and the finishing coat on top of that. I suppose it would cost about $1.50 per square yard, but it makes a first-class pavement.

In the city of Pittsburgh, ordinary asphalt pavement has a concrete foundation, an inch of asphalt binder, and two inches of asphalt wearing surface. The binder is put in to prevent the asphalt surface from creeping, although some authorities claim that it is safe to omit the asphalt binder and rough the surface of the concrete. Wherever the traffic is not heavy, an inch and a half of asphalt wearing surface is sufficient.

REPORT OF COMMITTEE ON ROADWAYS, SIDEWALKS AND FLOORS.

The Committee presents the following specifications for consideration and adoption.*

Proposed Revised Standard Specifications for Portland Cement Sidewalks.

Proposed Revised Standard Specifications for Concrete Roads and Street Pavements.

Proposed Revised Standard Specifications for Concrete Curb and Concrete Curb and Gutter.

Proposed Specifications for Plain Concrete Floors.

Proposed Specifications for Reinforced Concrete Floors.

Respectfully submitted,

C. W. Boynton, *Chairman.*
A. C. Birnie,
J. B. Landfield,
C. R. Miller,
A. E. Snodgrass.

* The specifications appear in the following pages as amended and adopted by letter ballot May 15, 1912.—Ed.

NATIONAL ASSOCIATION OF CEMENT USERS.

PHILADELPHIA, PA.

STANDARD SPECIFICATIONS FOR PORTLAND CEMENT SIDEWALKS.

REVISED MAY 15, 1912.

MATERIALS.

1. The cement shall meet the requirements of the Standard Specifications for Portland Cement of the American Society for Testing Materials and adopted by this Association. (Standard No. 1.) **Cement.**

2. The aggregates shall be clean, coarse, hard, durable materials and shall be free from dust, soft, flat or elongated particles, loam, vegetable or other deleterious matter. In no case shall aggregate containing frost or lumps of frozen material be used. **Aggregates.**

(a) *Fine Aggregate.*—Fine aggregate shall consist of sand, crushed stone or gravel screenings, preferably of silicious material, graded from fine to coarse and passing when dry, a screen having one-quarter (¼) inch diameter holes, and not more than three (3) per cent. shall pass a sieve having one hundred (100) meshes per linear inch.

Fine aggregate shall be of such quality that mortar composed of one (1) part Portland cement and three (3) parts fine aggregate by weight, when made into briquettes will show a tensile strength at least equal to the strength of 1 : 3 mortar of the same consistency made with the same cement and Standard Ottawa sand.

(b) *Coarse Aggregate.*—Coarse aggregate shall consist of inert materials such as crushed stone or gravel, graded in size, retained on a screen having one-quarter (¼) inch diameter holes and the maximum size shall be such as to pass a one and one-quarter (1¼) inch ring.

(c) *Natural Mixed Aggregates.*—Natural mixed aggregates shall not be used as they come from the deposit, but shall be screened and remixed to agree with the proportions specified.

Sub-Base. 3. Only clean, hard, suitable material, not exceeding four (4) inches in the largest dimension shall be used in the sub-base.*

Water. 4. Water shall be clean, free from oil, acid, alkali or vegetable matter.

Coloring. 5. If artificial coloring material is required, only mineral colors shall be used.

Reinforcement. 6. The reinforcing metal shall meet the requirements of the Standard Specifications for Steel Reinforcement adopted March 16, 1910, by the American Railway Engineering Association.

Joint Filler. 7. The expansion joint filler shall be a suitable elastic waterproof compound that will not become soft and run out in hot weather, nor hard and brittle and chip out in cold weather.

SUB-GRADE.*

Sub-Grade. 8. *Slope.*—The sub-grade shall have a slope toward the curb of not less than one-half (½) inch per foot.

9. *Depth.**—(a) The sub-grade shall not be less than eleven (11) inches below the finished surface of the walk.

(b) The sub-grade shall not be less than five (5) inches below the finished surface of the walk.

10. *Preparation.*—All soft and spongy places shall be removed and all depressions filled with suitable material which shall be thoroughly compacted in layers not exceeding six (6) inches in thickness.

11. *Deep Fills.*—When a fill exceeding one (1) foot in thickness is required to bring the work to grade, it shall be made in a manner satisfactory to the engineer. The top of all fills shall extend beyond the walk on each side at least one (1) foot, and the sides shall have a slope not greater than one (1) on one and one-half (1½).

Drainage. 12. *Drainage.*—When required, a suitable drainage system shall be installed and connected with sewers or other drains indicated by the engineer.

* NOTE.—When a sub-base is required, eliminate paragraph 9(b). When a sub-base is not required, eliminate paragraphs 3 and 9(a), 13 and 14. Unless paragraph 9(a) is eliminated, 9(b) is void.

SUB-BASE.*

13. *Width—Thickness.*—On the sub-grade shall be spread a suitable material as hereinbefore stated which shall be thoroughly rolled or tamped to a surface at least five (5) inches below the finished grade of the walk. On the fills, the sub-base shall extend the full width of the fill and the sides shall have the same slope as the sides of the fill. **Sub-Base.**

14. *Wetting.*—While compacting the sub-base, the material shall be kept thoroughly wet and shall be in that condition when the concrete is deposited.

FORMS.

15. *Materials.*—Forms shall be free from warp and of sufficient strength to resist springing out of shape. **Forms.**

16. *Setting.*—The forms shall be well staked or otherwise held to the established lines and grades and their upper edges shall conform to the established grade of the walk.

17. *Treatment.*—All wood forms shall be thoroughly wetted and metal forms oiled before depositing any material against them. All mortar and dirt shall be removed from forms that have been previously used.

CONSTRUCTION.

18. *Size of Slabs.*—The slabs or independently divided blocks when not reinforced shall have an area of not more than thirty-six (36) square feet and shall not have any dimension greater than six (6) feet. Larger slabs shall be reinforced as hereinafter specified. **Slabs.**

19. *Thickness of Walk.*—The thickness of the walks should not be less than five (5) inches for residence districts, and not less than six (6) inches for business districts.

20. *Width and Location of Joints.*—A one-half (½) inch expansion joint shall be provided at least once in every fifty (50) feet. **Joints.**

21. *Protection of Edges.*—Unless protected by metal, the upper edges of the concrete shall be rounded to a radius of one-half (½) inch. **Edges.**

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* NOTE.—When a sub-base is required, eliminate paragraph 9(*b*). When a sub-base is not required, eliminate paragraphs 3 and 9(*a*), 13 and 14. Unless paragraph 9(*a*) is eliminated, 9(*b*) is void.

MEASURING AND MIXING.

Measuring. 22. *Measuring.*—The method of measuring the materials for the concrete, including water, shall be one which will insure separate uniform proportions at all times. A bag of Portland cement (94 lb. net) shall be considered one (1) cubic foot.

Mixing. 23. The ingredients of the concrete or mortar shall be thoroughly mixed dry, sufficient water added to obtain the desired consistency, and the mixing continued until the materials are uniformly distributed and the mass is uniform in color and homogeneous.

(*a*) *Machine Mixing.*—When the conditions will permit, a machine mixer of a type that insures the uniform proportioning of the materials throughout the mass, shall be used.

(*b*) *Hand Mixing.*—When it is necessary to mix by hand, the mixing shall be on a water-tight platform and the materials shall be turned until the mass is uniform in color and homogeneous.

Retempering. 24. *Retempering*, that is, remixing mortar or concrete that has partially hardened with additional water, will not be permitted.

TWO-COURSE WALK.

BASE.

Two-Course Walk. Base. 25. *Proportions.*—The concrete shall be mixed in the proportion by volume of one (1) part Portland cement, two and one-half ($2\frac{1}{2}$) parts fine aggregate and five (5) parts coarse aggregate.

26. *Consistency.*—The materials shall be mixed wet enough to produce a concrete of a consistency that will flush readily under slight tamping, but which can be handled without causing a separation of the coarse aggregate from the mortar.

27. *Placing.*—After mixing, the concrete shall be handled rapidly and the successive batches deposited in a continuous operation completing individual sections. Under no circumstances shall concrete be used that has partially hardened. The forms shall be filled and the concrete struck off and tamped to a surface the thickness of the wearing course below the established grade of the walk. After the concrete has been thoroughly tamped against the cross forms, they shall be removed and the material or the adjoining slab deposited so as to preserve the joint. Workmen shall not be permitted to walk on the freshly

laid concrete, and if sand or dust collects on the base it shall be carefully removed before the wearing course is applied.

28. Slabs having an area of more than thirty-six (36) square feet, or having any dimension greater than six (6) feet, shall be reinforced with wire fabric or with plain or deformed bars. The cross-sectional area of metal shall amount to at least 0.041 sq. in. per lin. ft. The reinforcement shall be placed upon and slightly pressed into the concrete base immediately after the base is placed. Reinforcement shall not cross joints and shall be lapped sufficiently to develop the strength of the metal. **Reinforcement.**

WEARING COURSE.

29. *Proportions.*—The mortar shall be mixed in the manner hereinbefore specified in the proportion by volume of one (1) part Portland cement, and not more than two (2) parts fine aggregate. **Wearing Course.**

30. *Consistency.*—The mortar shall be of a consistency that will not require tamping, but which can be easily spread into position.

31. *Thickness.*—The wearing course of the walk in residence districts shall have a minimum thickness of three-quarters ($\frac{3}{4}$) of an inch, and in business districts a minimum thickness of one (1) inch.

32. *Placing.*—The wearing course shall be placed immediately after mixing and in no case shall more than fifty (50) minutes elapse between the time the concrete for the base is mixed and the time the wearing course is placed.

33. *Finishing.*—After the wearing course has been brought to the established grade, it shall be worked with a wood float in a manner that will thoroughly compact it. When required, the surface shall be troweled smooth, but excessive working with a steel trowel shall be avoided. The slab markings shall be made in the wearing course directly over the joints in the base with a tool which will completely separate the wearing course of adjacent slabs. If excessive moisture occurs on the surface, it must be taken up with a rag or mop and in no case shall dry cement or a mixture of dry cement and sand be used to absorb this moisture or to hasten the hardening. Unless protected by metal, the sur-

face edges of all slabs shall be rounded to a radius of about one-half (½) inch.

Coloring. 34. If artificial coloring is used, it must be incorporated with the entire wearing course and shall be mixed dry with the cement and aggregate until the mixture is of uniform color. In no case shall the amount of coloring used exceed five (5) per cent. of the weight of the cement.

ONE-COURSE WALK.

One-Course Walk. The general requirements of the specifications covering two-course work will apply to one-course work with the following exceptions:

35. *Proportions.*—The concrete shall be mixed in the proportion of one (1) part Portland cement, one and one-half (1½) parts fine aggregate and three (3) parts course aggregate passing a one (1) inch ring.

36. *Placing and Finishing.*—The forms shall be filled, the concrete struck off and the coarse particles forced back from the surface, and the work finished as specified under Two-Course Work.

37. *Reinforcement.*—When a single course walk is to be reinforced, the metal shall be placed two (2) inches from the finished surface. The minimum amount of metal shall be as specified in Paragraph 28.

PROTECTION.

Protection. 38. *Treatment.*—When completed, the work shall be kept moist and protected from traffic and the elements for at least three days, and shall not be opened to traffic until the engineer so directs.

39. *Temperature below 35 degrees F.*—If at any time during the progress of the work the temperature is, or in the opinion of the engineer will within twenty-four (24) hours drop to thirty-five (35) deg. Fahrenheit, the water and aggregates shall be heated and precautions taken to protect the work from freezing for at least five (5) days. In no case shall concrete be deposited upon a frozen sub-grade or sub-base.

STANDARD No. 5.

NATIONAL ASSOCIATION OF CEMENT USERS.

PHILADELPHIA, PA.

STANDARD SPECIFICATIONS FOR CONCRETE ROADS AND STREET PAVEMENTS.

REVISED MAY 15, 1912.

MATERIALS.

1. The cement shall meet the requirements of the Standard Specifications for Portland Cement of the American Society for Testing Materials and adopted by this Association. (Standard No. 1.) **Cement.**

2. The aggregates shall be clean, coarse, hard, durable materials and shall be free from dust, soft, flat or elongated particles, loam, vegetable or other deleterious material. In no case shall aggregate containing frost or lumps of frozen material be used. **Aggregates.**

(a) *Fine Aggregate for Concrete.*—Fine aggregate shall consist of sand, crushed stone or gravel screenings preferably of silicious material, graded from fine to coarse and passing, when dry, a screen having one-quarter ($\frac{1}{4}$) inch diameter holes, and not more than three (3) per cent. shall pass a sieve having one hundred (100) meshes per linear inch.

Fine aggregate shall be of such quality that mortar composed of one (1) part Portland cement and three (3) parts fine aggregate, by weight, when made into briquettes, will show a tensile strength at least equal to the strength of 1:3 mortar of the same consistency made with the same cement and Standard Ottawa sand.

(b) *Aggregate for Wearing Course.*—The aggregate shall consist of screened gravel or stone screenings from granite or other close-grained durable rock sufficiently hard to scratch glass, mixed in the proportion of three (3) parts passing a one-half ($\frac{1}{2}$) inch ring and retained on a screen having one-quarter ($\frac{1}{4}$) inch diameter holes and two (2) parts passing a screen having one-

quarter (¼) inch diameter holes and retained on a screen having fifty (50) meshes per linear inch.

(c) *Coarse Aggregate for Concrete.*—Coarse aggregate shall consist of inert materials such as stone or gravel, graded in size, retained on a screen having one-quarter (¼) inch diameter holes and the maximum size shall be such as to pass a one and one-half (1½) inch ring.

(d) *Natural Mixed Aggregates.*—Natural mixed aggregates shall not be used as they come from deposits, but shall be screened and remixed to agree with the proportions specified.

Sub-Base. 3. Only clean, hard, suitable material, not exceeding four (4) inches in the largest dimension, shall be used in the sub-base.*

Water. 4. Water shall be clean, free from oil, acid, alkali, or vegetable matter.

Color. 5. If artificial coloring matter is required, only mineral colors shall be used.

Reinforcement. 6. The reinforcing metal shall meet the requirements of the Standard Specifications for Steel Reinforcement adopted March 16, 1910, by the American Railway Engineering Association.

Joint Filler. 7. The expansion joint filler for open joints shall be a suitable elastic waterproof compound that will not become soft and run out in hot weather, nor hard and brittle and chip out in cold weather.

SUB-GRADE.*

Sub-Grade. 8. *Section.*—The sub-grade shall have a rise at the center of not more than one-hundredth (1/100) the width of the pavement.

9. *Depth.**—(a) The sub-grade shall not be less than twelve (12) inches below the finished surface of the pavement.

(b) The sub-grade shall not be less than six (6) inches below the finished surface of the pavement.

10. *Preparation.*—All soft and spongy places shall be removed and all depressions filled with suitable material which shall be

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* NOTE.—When a sub-base is required, eliminate paragraph 9(b). When a sub-base is not required, eliminate paragraphs 3, 9(a), 13 and 14. Unless paragraph 9(a) is eliminated, 9(b) is void.

thoroughly compacted in layers not exceeding six (6) inches in thickness.

11. *Deep Fills.*—When a fill exceeding one (1) foot in thickness is required to bring the pavement to grade, it shall be made in a manner satisfactory to the engineer.

12. *Drainage.*—When required, a suitable drainage system shall be installed and connected with sewers or other drains indicated by the engineer. **Drainage.**

SUB-BASE.

13. *Thickness.*—On the sub-grade shall be spread a material as hereinbefore specified, which shall be thoroughly rolled and tamped to a surface at least six (6) inches below the finished grade of the pavement. **Sub-Base.**

14. *Wetting.*—While compacting the sub-base, the material shall be kept thoroughly wet and shall be in that condition when the concrete is deposited.

FORMS.

15. *Materials.*—Forms shall be free from warp and of sufficient strength to resist springing out of shape. **Forms.**

16. *Setting.*—The forms shall be well staked or otherwise held to the established lines and grades and their upper edges shall conform to the established grade of the pavement.

17. *Treatment.*—All wood forms shall be thoroughly wetted and metal forms oiled before depositing any material against them. All mortar and dirt shall be removed from forms that have been previously used.

EXPANSION JOINTS.

18. *Width and Location.*—Expansion joints not less than one-quarter (¼) inch nor more than one-half (½) inch in width shall be placed across the street or road not more than twenty-five (25) feet apart, perpendicular to the center line. When a curb or combination curb and gutter is used, a similar joint shall be placed between it and the pavement. All expansion joints shall extend through the entire thickness of the pavement. **Expansion Joints.**

19. *Protection of Edges.*—When required by the engineer in

charge, the expansion joints shall be protected with metal. Unless protected by metal, the upper edges of the concrete shall be rounded to a radius of one-half ($\frac{1}{2}$) inch.

MEASURING AND MIXING.

Measuring. 20. *Measuring.*—The method of measuring the materials for the concrete, including water, shall be one which will insure separate, uniform proportions at all times. A bag of Portland cement (94 lb. net) shall be considered one (1) cubic foot.

Mixing. 21. The ingredients of the concrete or mortar shall be thoroughly mixed dry, sufficient water added to obtain the desired consistency, and the mixing continued until the materials are uniformly distributed and the mass is uniform in color and homogeneous.

(*a*) *Machine Mixing.*—When the conditions will permit, a machine mixer of a type which insures the uniform proportioning of the materials throughout the mass, shall be used.

(*b*) *Hand Mixing.*—When it is necessary to mix by hand, the mixing shall be on a water-tight platform and the materials shall be turned until the mass is uniform in color and homogeneous.

Retempering. 22. *Retempering,* that is, remixing mortar or concrete that has partially hardened with additional water, will not be permitted.

TWO-COURSE PAVEMENT.

BASE.

Two-Course Work. 23. *Proportions.*—The concrete shall be mixed in the proportion by volume of one (1) part Portland cement, two and one-half ($2\frac{1}{2}$) parts fine aggregate and five (5) parts coarse aggregate.

Base. 24. *Consistency.*—The materials shall be mixed wet enough to produce a concrete of a consistency that will flush readily under light tamping, but which can be handled without causing a separation of the coarse aggregate from the mortar.

25. *Placing.*—After mixing, the concrete shall be handled rapidly and successive batches deposited in a continuous operation, completing individual sections. Under no circumstances shall concrete be used that has partially hardened. The concrete

shall be well tamped to a surface the thickness of the wearing course below the established grade of the pavement. Workmen shall not walk on the freshly laid concrete and if sand or dust collects on the base, it shall be carefully removed before the wearing course is applied.

Reinforcement.

26. On streets more than twenty-five (25) feet wide not having car tracks, the pavement shall be reinforced with wire fabric or with plain or deformed bars. The cross-sectional area of metal shall amount to at least 0.041 sq. in. per foot measured parallel to the axis of the street and at least 0.025 sq. in. per foot measured perpendicular to the axis of the street. The reinforcement shall be placed upon and slightly pressed into the concrete base immediately after the base is placed. Reinforcement shall not cross expansion joints and shall be lapped sufficiently to develop the strength of the metal.

WEARING COURSE.

Wearing Course.

27. *Proportions.*—The mortar shall be mixed in the manner hereinbefore specified in the proportion by volume of one (1) part Portland cement and not more than two (2) parts aggregate for wearing course.

28. *Consistency.*—The mortar shall be of a consistency that will not require tamping but which can be easily spread into position with a template or straight-edge.

29. *Thickness.*—The wearing course of the pavement in residence districts shall have a minimum thickness of one and one-half ($1\frac{1}{2}$) inches and in business districts a minimum of two (2) inches.

30. *Placing.*—The wearing course shall be placed immediately after mixing, and in no case shall more than fifty (50) minutes elapse between the time the concrete for the base is mixed and the time the wearing course is placed.

31. *Finishing.*—After the wearing course has been brought to the established grade, it shall be worked with a wood float in a manner that will thoroughly compact it. Before the wearing course has completely hardened, it shall be roughened by brushing with a stiff vegetable fibre brush or broom. On grades of over five (5) per cent. the surface shall be corrugated if directed by the engineer.

Coloring.

32. If artificial coloring is used, it must be incorporated with the entire wearing course and shall be mixed dry with the cement and aggregate until the mixture is of uniform color. In no case shall the amount of coloring used exceed five (5) per cent. of the weight of the cement.

ONE-COURSE PAVEMENT.

One-Course Work.

The general requirements of the specifications covering two-course work will apply to one-course work with the following exceptions:

33. *Proportions.*—The concrete shall be mixed in the proportion by volume of one (1) part Portland cement to one and one-half ($1\frac{1}{2}$) parts fine aggregate (Paragraph 2 *a*) or aggregate for wearing course (Paragraph 2 *b*) and three (3) parts coarse aggregate passing a one (1) inch ring.

34. *Placing and Finishing.*—The concrete shall be placed as provided for under Two Course Pavement, tamped and struck off to grade. The coarse particles shall then be forced to a depth below the surface which will permit of finishing as specified under Two Course Pavement.

35. *Reinforcement.*—When a one-course pavement is to be reinforced, the metal shall be placed two (2) inches from the upper surface of the pavement. The minimum amount of metal shall be as specified under Two Course Pavement.

WEARING SURFACE OF BITUMEN AND SAND.

Wearing Surface of Bitumen and Sand.

36. *Proportions.*—When a wearing surface of bitumen and sand is used, it shall be placed upon a one-course pavement constructed as hereinbefore specified.

37. *Bitumen.*—The bitumen shall be of a quality specified by the engineer.

38. *Placing.*—After the concrete base has hardened for at least seven (7) days, the clean, dry surface of the pavement shall be covered with hot bitumen applied with a sprinkling wagon designed for the purpose, or with suitable hand sprinkling cans. The hot bitumen shall be evenly distributed over the concrete by brushing with suitable brooms immediately after applying, and then covered with the fine aggregate. (Paragraph 2 *b*.)

39. *Amount of Bitumen and Sand.*—Between one-third ($\frac{1}{3}$) and one-half ($\frac{1}{2}$) gallon of bitumen shall be applied per square yard of pavement. Approximately one (1) cubic yard of fine aggregate shall be applied per one hundred and fifty (150) square yards of pavement.

40. *Expansion Joints.*—Before applying the wearing surface of bitumen and sand, all expansion joints in the pavement shall be filled as hereinbefore specified. Where a wearing surface of bitumen and sand is used, the edge of the expansion joint need not be protected with metal.

CROWN.

41. *Amount.*—All types of concrete pavement shall be given a rise or crown at the center of at least one-hundredth (1/100) but not more than one seventy-fifth (1/75) of the width of the pavement. A portion of this crown may be obtained by increasing the thickness of the pavement at the center rather than by laying a pavement of uniform thickness on a crowned sub-grade or sub-base. **Crown.**

PROTECTION.

42. *Treatment.*—When completed, the work shall be kept moist and protected from the elements for at least three (3) days, and the pavement shall not be open to traffic until the engineer so directs. **Protection.**

43. *Temperature below* 35° *F.*—If at any time during the progress of the work the temperature is, or in the opinion of the engineer will, within 24 hours, drop to thirty-five (35) degrees Fahrenheit, the water and aggregates shall be heated and precautions taken to protect the work from freezing for at least five (5) days. In no case shall concrete be deposited upon a frozen sub-grade or sub-base.

NATIONAL ASSOCIATION OF CEMENT USERS.

PHILADELPHIA, PA.

STANDARD SPECIFICATIONS FOR CONCRETE CURB AND CONCRETE CURB AND GUTTER.

REVISED MAY 15, 1912.

MATERIALS.

Cement. 1. The cement shall meet the requirements of the Standard Specifications for Portland Cement of the American Society for Testing Materials and adopted by this Association. (Standard No. 1.)

Aggregates. 2. The aggregates shall be clean, coarse, hard, durable materials and shall be free from dust, soft, flat or elongated particles, loam, vegetable or other deleterious matter and in no case shall aggregates containing frost or lumps of frozen material be used.

(*a*) *Fine Aggregate.*—Fine aggregate shall consist of sand, crushed stone or gravel screenings, preferably of silicious material, graded from fine to coarse and passing when dry a screen having one-quarter ($\frac{1}{4}$) inch diameter holes, and not more than three (3) per cent. shall pass a sieve having one hundred (100) meshes per linear inch.

Fine aggregate shall be of such quality that mortar composed of one (1) part Portland cement and three (3) parts fine aggregate by weight when made into briquettes, will show a tensile strength at least equal to the strength of 1 : 3 mortar of the same consistency made with the same cement and Standard Ottawa sand.

(*b*) *Coarse Aggregate.*—Coarse aggregate shall consist of inert materials such as crushed stone or gravel graded in size, retained on a screen having one-quarter ($\frac{1}{4}$) inch diameter holes, and the maximum size shall be such as to pass a one and one-quarter ($1\frac{1}{4}$) inch ring.

(*c*) *Natural Mixed Aggregates.*—Natural mixed aggregates shall not be used as they come from the deposit, but shall be screened and remixed to agree with the proportions specified.

3. Only clean, hard, suitable materials, not exceeding four (4) inches in the largest dimension shall be used in the sub-base. **Sub-Base.**

4. Water shall be clean, free from oil, acid, alkali, or vegetable matter. **Water.**

5. If artificial coloring material is required, only mineral colors shall be used. **Coloring.**

6. The expansion joint filler shall be a suitable, elastic, waterproof compound that will not become soft and run out in hot weather, nor hard and brittle and chip out in cold weather. **Joint Filler.**

SUB-GRADE.

7. *Depth below Grade.* (*a*) *Concrete Curb.*—When a sub-base is required, the sub-grade shall not be less than thirty (30) inches below the established grade of the curb. **Depth.**

(*b*) *Concrete Curb and Gutter.*—When a sub-base is required, the sub-grade shall not be less than eleven (11) inches below the established grade of the gutter.

8. *Preparation.*—All soft and spongy places shall be removed and all depressions filled with suitable material which shall be thoroughly compacted in layers not exceeding six (6) inches in thickness. **Fills.**

9. *Deep Fills.*—When a fill exceeding one (1) foot in thickness is required to bring the work to grade, it shall be made in a manner satisfactory to the engineer.

10. *Drainage.*—When required, a suitable drainage system shall be installed and connected with sewers or other drains indicated by the engineer. **Drainage.**

SUB-BASE.

11. *Thickness.* (*a*) *Concrete Curb.*—On the sub-grade shall be spread a material as heretofore specified, which shall be thoroughly rolled or tamped to a surface at least twenty-four (24) inches below the established grade of the curb. **Thickness.**

(*b*) *Concrete Curb and Gutter.*—On the sub-grade shall be spread a material as heretofore specified, which shall be thoroughly rolled or tamped to a surface at least six (6) inches below the established grade of the gutter.

12. *Wetting.*—While compacting the sub-base, the material shall be kept thoroughly wet and shall be in that condition when the concrete is deposited.

FORMS.

Forms.

13. *Materials.*—Forms shall be free from warp and of sufficient strength to resist springing out of shape.

14. *Setting.*—The forms shall be well staked or otherwise held to the established lines and grades, and their upper edges shall conform to the established grade of the curb or curb and gutter.

15. *Treatment.*—All wood forms shall be thoroughly wetted and metal forms oiled before depositing any material against them. All mortar and dirt shall be removed from forms that have been previously used.

CONSTRUCTION.

Curb.

16. *Dimension of Curb.*—The section of the curb shall conform with that shown in Fig. 1. The thickness at the base shall not be less than twelve (12) inches and at the top not more than six (6) inches, with a batter on the street side of one (1) to four (4).

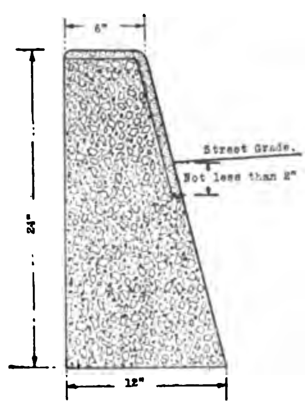

FIG. 1.

Curb and Gutter.

17. *Dimensions of Curb and Gutter.*— The section of the combination curb and gutter shall conform with that shown in

Fig. 2. The depth of the back of the curb shall not be less than twelve (12) inches and the depth of the face not less than six (6) inches. The breadth of the gutter shall not be less than sixteen (16) inches nor more than twenty-four (24) inches.

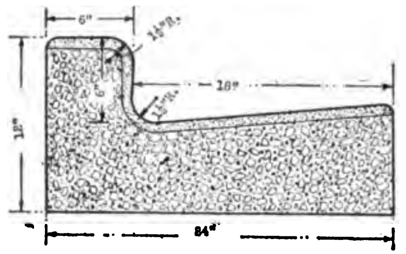

FIG. 2.

18. *Size of Sections.*—The curb and gutter shall be divided into sections not less than five (5) nor more than eight (8) feet long by some method which will insure the complete separation of the sections. Sections.

19. *Section at Street Corners.*—The construction of the combination curb and gutter at street corners shall conform with that

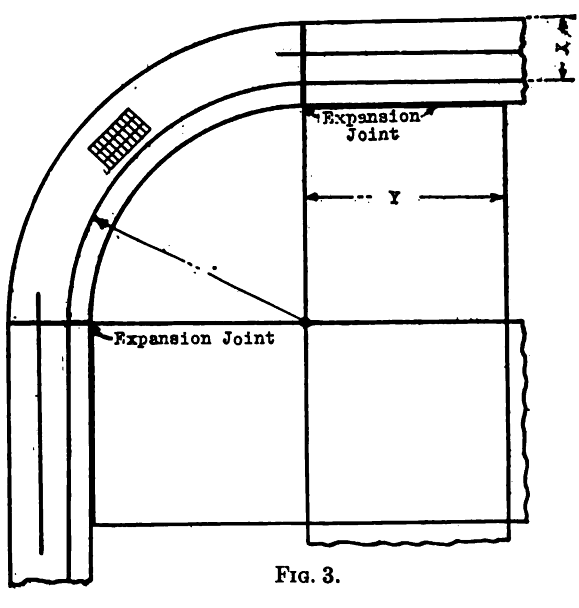

FIG. 3.

shown in Fig. 3. The radius of the curb shall not be less than six (6) feet.

Joints. 20. *Width and Location of Joints.*—A one-half (½) inch expansion joint shall be provided at least once in every one hundred and fifty (150) feet.

Edges. 21. *Protection of Edges.*—Unless protected by metal, the upper edges of the concrete shall be rounded to a radius of one-half (½) inch.

MEASURING AND MIXING.

Measuring. 22. *Measuring.*—The method of measuring the materials for the concrete, including water, shall be one which will insure separate uniform proportions at all times. A bag of Portland cement (94 lb. net) shall be considered one (1) cubic foot.

Mixing. 23. The ingredients of the concrete or mortar shall be thoroughly mixed dry, sufficient water added to obtain the desired consistency, and the mixing continued until the materials are uniformly distributed and the mass is uniform in color and homogeneous.

(*a*) *Machine Mixing.*—When the conditions will permit, a machine mixer of a type which insures the uniform proportioning of the materials throughout the mass shall be used.

(*b*) *Hand Mixing.*—When it is necessary to mix by hand, the mixing shall be on a water-tight platform and the materials shall be turned until the mass is uniform in color and homogeneous.

Retempering. 24. *Retempering*, that is remixing mortar or concrete that has partially hardened with additional water, will not be permitted.

TWO-COURSE CURB AND CURB AND GUTTER.

BASE.

Two-Course Work. 25. *Proportions.*—The concrete shall be mixed in the proportion by volume of one (1) part Portland cement, two and one-half (2½) parts fine aggregate, and five (5) parts coarse aggregate.

Base.

26. *Consistency.*—The materials shall be mixed wet enough to produce a concrete of a consistency that will flush readily under slight tamping, but which can be handled without causing a separation of the coarse aggregate from the mortar.

27. *Placing.*—After mixing, the concrete shall be handled rapidly and the successive batches deposited in continuous operation completing individual sections. Under no circumstances shall concrete be used that has partially hardened. The gutter forms shall be filled and the concrete struck off and tamped to a surface the thickness of the wearing course below the established grade of the gutter. The concrete for the curb shall be placed and tamped so as to permit of the application of the required wearing course to the face and top so as to bring the work to the established line and grade of the curb. The work shall be executed in a manner which will insure perfect joints between abutting sections. Workmen shall not be permitted to walk on freshly laid concrete, and if sand or dust collects on the base, it shall be carefully removed before the wearing course is applied.

WEARING COURSE.

28. *Proportions.*—The mortar shall be mixed in the manner hereinbefore specified in the proportion by volume of one (1) part Portland cement and not more than two (2) parts fine aggregate. **Wearing Course.**

29. *Consistency.*—The mortar shall be of a consistency that will not require tamping but which can be easily spread into position.

30. *Thickness.*—The wearing course of the gutter and top and face of the curb shall have a minimum thickness of three-quarters ($\frac{3}{4}$) of an inch.

31. *Placing.*—The wearing course shall be placed immediately after mixing, and in no case shall more than fifty (50) minutes elapse between the time the concrete for the base is mixed and the time the wearing course is placed.

32. *Finishing.*—After the wearing course has been brought to the established line and grade, it shall be worked with a wood float in a manner which will thoroughly compact it. When required, the surface shall be troweled smooth, but excessive working with a steel trowel shall be avoided. The section markings shall be made in the wearing courses directly over the joints in the base with a tool which will completely separate the wearing courses of adjacent sections. If excessive moisture occurs on the surface, it must be taken up with a rag or mop, and in no case

shall dry cement or a mixture of dry cement and sand be used to absorb this moisture or to hasten the hardening. The edge of the curb on the street side and the intersection of the curb and gutter shall be rounded to a radius of about one and one-half (1½) inches. All other edges shall be rounded to a radius of one-half (½) inch unless protected by metal.

Coloring. 33. If artificial coloring is used, it must be incorporated with the entire wearing course and shall be mixed dry with the cement and aggregate until the mixture is of uniform color. In no case shall the amount of coloring used exceed five (5) per cent. of the weight of the cement.

ONE-COURSE CURB AND CURB AND GUTTER.

One-Course Work. The general requirements of the specifications covering two-course work will apply to one-course work, with the following exceptions:

34. *Proportions.*—The concrete shall be mixed in the proportion by volume of one (1) part Portland cement, one and one-half (1½) parts fine aggregate and three (3) parts coarse aggregate passing a one (1) inch ring.

35. *Placing and Finishing.*—The forms shall be filled, the concrete struck off and the coarse particles forced back from the surface, and the work finished as specified under Two Course work.

PROTECTION.

Protection. 36. *Treatment.*—When completed the work shall be kept moist and protected from traffic and the elements for at least one (1) week.

37. *Temperature below 35 degrees F.*—If at any time during the progress of the work, the temperature is, or in the opinion of the engineer will within twenty-four (24) hours drop to thirty-five (35) deg. Fahrenheit, the water and aggregates shall be heated and precautions taken to protect the work from freezing for at least five (5) days. In no case shall concrete be deposited upon a frozen sub-grade or sub-base.

NATIONAL ASSOCIATION OF CEMENT USERS.

PHILADELPHIA, PA.

STANDARD SPECIFICATIONS FOR PLAIN CONCRETE FLOORS.

ADOPTED MAY 15, 1912.

MATERIALS.

1. The cement shall meet the requirements of the Standard Specifications for Portland Cement of the American Society for Testing Materials and adopted by this Association (Standard No. 1). **Cement.**

2. The aggregates shall be clean, coarse, hard, durable materials and shall be free from dust, soft, flat or elongated particles, loam, vegetable or other deleterious matter. In no case shall aggregate containing frost or lumps of frozen material be used. **Aggregates.**

(a) *Fine Aggregate for Concrete.*—Fine aggregate shall consist of sand, crushed stone or gravel screenings, preferably of silicious material, graded from fine to coarse and passing, when dry, a screen having one-quarter ($\frac{1}{4}$) inch diameter holes, and not more than three (3) per cent. passing a sieve having one hundred (100) meshes per linear inch.

Fine aggregate shall be of such quality that mortar composed of one (1) part Portland cement and three (3) parts fine aggregate by weight when made into briquettes will show a tensile strength at least equal to the strength of 1:3 mortar of the same consistency made with the same cement and Standard Ottawa sand.

(b) *Aggregate for Wearing Course.*—The aggregate shall consist of screened gravel or stone screenings from granite or other close-grained, durable rock, sufficiently hard to scratch glass, mixed in the proportion of three (3) parts passing a one-half ($\frac{1}{2}$) inch ring and retained on a screen having one-quarter ($\frac{1}{4}$) inch diameter holes, and two (2) parts passing a screen having one-quarter ($\frac{1}{4}$) inch diameter holes and retained on a screen having fifty (50) meshes per linear inch.

(c) *Coarse Aggregate for Concrete.*—Coarse aggregate shall consist of inert materials, such as crushed stone or gravel, graded in size, retained on a screen having one-quarter ($\frac{1}{4}$) inch diameter holes, and the maximum size shall be such as to pass a one and one-quarter ($1\frac{1}{4}$) inch ring.

(d) *Natural Mixed Aggregates.*—Natural mixed aggregates shall not be used as they come from the deposit but shall be screened and remixed to agree with the proportions specified.

Water. 3. Water shall be clean, free from oil, acid, alkali, or vegetable matter.

Coloring. 4. If artificial coloring material is required, only mineral colors shall be used.

Reinforcement. 5. The reinforcement shall meet the requirements of the Standard Specifications for Steel Reinforcement adopted March 16, 1910, by the American Railway Engineering Association.

CONSTRUCTION.

Forms. 6. The forms shall be substantial, unyielding and so constructed that the concrete will conform to the designed dimensions and contours and shall also be tight to prevent the leakage of mortar. The supports for floors shall not be removed in less than ten (10) days after the concrete is placed, and then only with the consent of the engineer in charge.

Reinforcement. 7. The reinforcement shall be free from rust, scale or coatings of any character which will tend to reduce or destroy the bond, and shall be placed and held in position so that it will not become disarranged during the depositing of the concrete. Whenever it is necessary to splice tension reinforcement, the character of the splice shall be such as will develop its full strength. Splices at the point of maximum stress shall be avoided.

MEASURING AND MIXING.

Measuring. 8. The method of measuring the materials for the concrete, including water, shall be one which will insure separate, uniform proportions at all times. A bag of Portland cement (94 lb. net) shall be considered one (1) cubic foot.

Mixing. 9. The ingredients of the concrete or mortar shall be thoroughly mixed dry, sufficient water added to obtain the desired

consistency and the mixing continued until the materials are uniformly distributed and the mass is uniform in color and homogeneous.

(a) *Machine Mixing.*—When the conditions will permit, a machine mixer of a type which insures the uniform proportioning of the materials throughout the mass, shall be used.

(b) *Hand Mixing.*—When it is necessary to mix by hand, the mixing shall be on a water-tight platform and the materials shall be turned until the mass is uniform in color and homogeneous.

10. *Retempering,* that is, remixing mortar or concrete that has partially hardened with additional water, will not be permitted. **Retempering.**

TWO-COURSE FLOORS.

11. *Proportions.*—The concrete shall be mixed in the proportion by volume of one (1) part Portland cement, two (2) parts fine aggregate and four (4) parts coarse aggregate. **Slabs.**

12. *Consistency.*—The materials shall be mixed wet enough to produce a concrete of a consistency that will flow into the forms and about the reinforcement, but which can be conveyed from the mixer to the forms without the separation of the coarse aggregate from the mortar.

13. *Placing.*—When it is necessary to make a joint in a floor slab, its location shall be designated by the engineer. The concrete shall be placed in a manner to insure a smooth ceiling, and thoroughly worked around the reinforcement and into the recesses of the forms. It shall be brought to a surface at least one (1) inch below the finished surface of the floor and as soon as the slab has hardened sufficiently to permit of workmen being upon it, and before it has dried out, the wearing course shall be applied. Workmen shall not be permitted to walk on freshly laid concrete, and if sand or dust collects on the base, it shall be carefully removed before the wearing course is applied.

14. *Proportions.*—The mortar shall be mixed in the manner hereinbefore specified in the proportion of one (1) part Portland cement and not more than two (2) parts aggregate for wearing course. **Wearing Course.**

15. *Consistency.*—The mortar shall be of a consistency that water will flush to the surface only under heavy, thorough working with a wood float.

16. *Thickness.*—The wearing course of the floor shall have a minimum thickness of one (1) inch.

17. *Placing.*—The wearing course shall be placed immediately after mixing.

18. *Finishing.*—After the wearing course has been brought to the established grade, it shall be worked with a wood float in a manner which will thoroughly compact it. When required, the surface shall be troweled smooth, but excessive working with a steel trowel shall be avoided. If excessive moisture occurs on the surface, it must be taken up with a rag or mop, and in no case shall dry cement or a mixture of dry cement and sand be used to absorb this moisture or to hasten the hardening.

19. *Coloring.*—If artificial coloring is used, it must be incorporated with the entire wearing course and shall be mixed dry with the cement and aggregate until the mixture is of uniform color. In no case shall the amount of coloring exceed five (5) per cent. of the weight of the cement.

WEARING SURFACE ON HARDENED SLABS.

The general requirements of the specifications covering two-course work will apply to Wearing Surface on Hardened Slabs with the following exceptions:

Preparation of Slab.

20. All dirt and debris shall be removed from the surface of the slab and the surface roughened in a manner that will expose the freshly broken face of the coarse aggregate. The floor shall be thoroughly scrubbed with a warm soap, lye, or acid solution, and rinsed with clear water, preferably under pressure. All water used to clean the slab shall be taken up with a rag or mop or otherwise disposed of.

21. *Treatment of Slab.*—The slab shall be thoroughly wet at the time the wearing course is placed. The surface of the slab shall be carefully treated with a neat cement grout brushed on, or with such a preparation as the engineer may direct.

Wearing Course.

22. *Consistency.*—The mortar for the lower one-quarter ($\frac{1}{4}$) inch shall be of such a consistency that it can be easily spread

with a straight edge. The mortar for the remainder of the wearing surface shall be of the consistency specified for wearing surface.

23. *Placing.*—The wearing surface shall be placed in two operations. Upon the freshly placed cement grout, a one-quarter (¼) inch layer of mortar of the consistency specified, shall be placed and thoroughly worked into the irregularities of the surface of the slab, and immediately followed with sufficient mortar of the consistency specified for wearing course to bring the floor to the established grade.

PROTECTION.

24. *Treatment.*—When completed, the floor shall be covered with at least two (2) inches of wet sand or sawdust which shall be kept moist for four (4) days and no one shall be allowed on the floor during that time. **Protection.**

25. *Temperature below 35 degrees F.*—If at any time during the progress of the work the temperature is, or in the opinion of the engineer will within twenty-four (24) hours drop to 35 deg. Fahrenheit, the water and aggregate shall be heated and precautions taken to protect the work from freezing for at least five days. In no case shall concrete be deposited upon a frozen base.

DISCUSSION

Mr. Boynton.

MR. C. W. BOYNTON.—There is probably nothing in the line of concrete work that is bothering everyone who has anything to do with concrete floors more just now than the question of dusting.

Our committee has undertaken to write a specification, which, if carefully followed, it is believed will reduce the amount of dusty floors, but it will not eliminate them. There are many conditions that affect results which seem to be practically beyond control as the art of floor laying is understood at this time.

The specifications do not cover the design of the floor but simply the method of placing the materials and the finishing and putting on the wearing surface. In both the plain and reinforced concrete floors it is advocated to place the wearing course immediately following the placing of the slab proper. In most cases this will require a change in the method of carrying out the work.

The specifications advocate placing the materials rather dry. The theory is that the material that is giving trouble consists of fine particles that are practically dead or inert, and are floated to the surface with the excess water. By using a drier mixture and working it down hard, less of the fine inert particles will be floated to the surface, and consequently less dusting will result. I think the theory is good and believe that if clean materials are used a large part of the trouble from dusting will be avoided.

Another practice that results in dusting is that of troweling the surface too much. If floors could be laid with a minimum or possibly without any repetition of troweling over the same area, it is probable that there would be a good deal less dusting. A finish course cannot be tamped with a heavy tamper, but the same result can be obtained with a wooden float or some such tool. We advocate just enough troweling with a trowel to give the surface the required finish. In some work a steel trowel would not be necessary at all. In other places it would be, according to the use to which the surface is to be put.

Mr. Belford.

MR. A. BELFORD.—Some of the large contractors who have laid very great areas of reinforced floors in two layers, have noticed

trouble arising from the fact that the lower layer was not thoroughly wet or saturated before the top layer was put on. They hold that the bottom layer, not being sufficiently saturated with water, absorbs the water from the top layer before it is thoroughly set, and there is not sufficient left to completely harden or give the top layer a sufficient amount of water for full crystallization. I have made tests and know that some cement must have water for at least 4 days in order to get all the water that is required for crystallization. That is, a cement that will take 3 hours for the initial set and say 6 for the final. **Mr. Belford.**

MR. BOYNTON.—I think Mr. Belford might put his statement in a different way and require that work be protected for a given length of time from the loss of any of the moisture that it originally contained. I do not believe that by sprinkling one can make up for the lack of sufficient water in the mixture. The aim should be to have enough water in the mass to bring about the necessary action and then keep the water there. **Mr. Boynton.**

If the top is placed in two layers there is danger of the upper portion being robbed of some of its moisture, especially if a dry top, as is considered good practice for the purpose of avoiding dusting, is used. If the first layer is so dry that it is going to absorb moisture from the second and the top is also subjected to evaporation, it would seem that a worse condition would exist than prevails with the now common method of placing the top excessively wet.

The theory is that enough moisture, possibly an excess of moisture over that necessary to bring about proper hardening, will be used, but the moisture must be retained, for if it is allowed to evaporate or is absorbed the top will be weak.

I think a great deal of the dust comes from the fine aggregate; there is entirely too much carelessness about the character of the materials used in wearing surfaces. Stone dust or particles of clay, which are very apt to come with the sand, or even fine silicious particles, will naturally work to the top. Also a certain amount of inert material comes out of the cement and the more the top is worked the more inert material will be developed. There is a certain portion of the cement, that is the very fine part, which is readily soluble and will separate itself from the remainder in the process of working. It is carried to the top with the excess

Mr. Boynton. moisture and when it dries out will readily dust. When the top is worked up and troweled excessively, these particles of cement are agitated in the excess moisture, and every stroke of the trowel helps to destroy the active setting quality of this cement. This inert material from the aggregate and cement is laitance. The finer the cement is ground the greater the tendency to rapid hydration and to the formation of laitance.

A Member. A MEMBER.—In a majority of cases the fine dust that is troweled to the top is from the cement. This has been shown by repeated tests and chemical analyses; the latter, knowing the character of the aggregate, have nearly always pointed to the very fine particles of the cement.

With a very moist top there is almost a psychological moment when the finishing ought to be done. It can only be gauged by watching carefully so as to obviate this ultra-troweling to bring the concrete to the top, which, assuming that all materials are satisfactory, is probably in nine-tenths of the cases the cause of the dusting.

With the same amount of troweling I think there is much more likelihood of bringing more dead material to the surface from a finely ground cement than from a coarse cement. Of course cement is not absolutely uniform. There are bound to be certain parts of it a little lower in specific gravity; that is, every particle has not the same specific gravity as every other particle; and in fine grinding there is probably some part of it a little less hard burned, a little bit lighter, which is also more easily pulverized. I believe the finer ground the cement is the more chance there is for bringing more of that dust to the top; and it does not set up by itself nearly as hard as it does with something larger to bind to.

Mr. Talbot. MR. K. H. TALBOT—In starting on the investigation of dusting of concrete floors, we made an inspection of a number of floors in the vicinity of Chicago, and found that there was a considerable amount of trouble due to dusting. This dusting varied from a very little to sufficient to have a detrimental effect on machinery.

In order to ascertain, if possible, the best manner of placing a concrete floor so that it would not prove dusty, I made up a set of experimental slabs using between 10 and 15 per cent of the total weight of the mixture of water. Those slabs in which the concrete used contained only 10 or 12 per cent of water appeared upon

grinding with a palmetto brush and carbon disk, to show less dusting than those which were made with more water and which were troweled longer. In the laboratory one of the slabs became wet before it was completed, and it was necessary to go back to it after thirty minutes and trowel it a second time in order to get rid of the excess moisture. This slab showed more dusting than any of the others. This dusting consisted of a deeper coat of fine material on top of the slab than was noticeable in the drier slab. Later all these slabs were ground with carborundum stone and the result was a very pleasing surface, especially in the case where the aggregate for the top had been a mixture of sand and granite screenings passing a $\frac{1}{4}$-in. and retained on a No. 50 screen. Mr. Talbot.

The question of obtaining a satisfactory bond between old slabs and the top was taken up by building a number of experimental slabs, some of which had their edges turned up in order to keep the original quantity of water in the mortar. Some of the architects and contractors in the vicinity of Chicago felt that as long as under practical conditions there was a considerable amount of water which stood in the top course and had to evaporate, that the cement did not get a chance to harden satisfactorily. Our results showed that all these tops which were so placed hardened up satisfactorily, and that a good bond between the old slab and top could be obtained by carefully cleaning and roughening the old slab.

We have had some experience with floors which were laid on structural slabs after the building was completed, and which were placed so wet that from three to twelve hours passed after placing before the finishers were able to leave it. Both of these floors have dusted badly, and it would seem that a part of the trouble had been due to troweling after the cement had taken its initial set, thereby disturbing the hardening process and reducing the final strength of the concrete.

A very interesting case occurred in the Liquid Carbonic Company's building in Chicago. The middle panel was comparatively soft and chalky, while the remainder of the floor was exceptionally good. It seems that in placing these floors, the workmen had worked from two directions and had proceeded toward the center, carrying such laitance and water as had once

Mr. Talbot. come off of the slabs during their work before them, and that this material was allowed to remain on the center slab.

Mr. Boynton. MR. BOYNTON.—Our specifications provide for removing any excess moisture that might show up in the work. By the ordinary methods any excess of moisture accumulates on the top and instead of troweling that along on to the next slab or portion of the work that is being laid, we provide for its removal. We believe this will remove the laitance that is developed. The specifications also forbid the use of driers, either as driers or as a means of hastening the hardening, which is practically the same thing so far as the result is concerned.

Mr. Northcott. MR. B. H. NORTHCOTT.—I would like to know exactly what is meant by clean sand.

Mr. Boynton. MR. BOYNTON.—We mean, for instance, a fine aggregate as specified for wearing course containing nothing that would pass through a No. 50 sieve. This eliminates all the fine loose particles that would sift out, but if the particles of the aggregate were coated in any way with material that could not be separated out by the sieve, such aggregate should be rejected. As a suggestion for determining this, I would say that any sand which, when put into a fruit jar or glass of water and stirred up, would show enough suspended matter to make the liquid decidedly cloudy, would be bad material for a top course.

I would not consider it safe to use a bituminous material as a bond or waterproofing between the bottom and top course unless purchased under a definite specification. In studying this subject during the past year the Committee obtained some bonding material from one of the concerns located in Chicago which is putting such material on the market. As a severe test it was put on the top of a slab to bind the top course to the slab. It developed no bond whatever. If a heavy top is put on a slab there would be no opportunity to determine the degree of bond established unless some occasion required cutting through the slab at a future time. Therefore, I do not believe it would be wise to use a tar or asphalt binder until it is known definitely what the material will do.

Mr. Brett. MR. ALLEN BRETT.—As far as bonding is concerned and in regard to using a coating between top and bottom, it just occurs to me to ask why it is necessary to have a bond? We have really

two materials, a concrete base and a top finish. They have a different coefficient of expansion. We make an endeavor to bond those two materials where we really can never actually bond them, because they have a different coefficient of expansion. So what is the need of a bond? The top rests right on the bottom and if it lies right there on that asphalt binder I think that it would stay there just as long as you need it there. Mr. Brett.

MR. BOYNTON.—Mr. Brett's remarks might be all right from a theoretical standpoint, but we do want a bond and we do get a bond regardless of this varying coefficient of expansion between top and bottom. It is known that when a bond is not obtained between the top and base of a sidewalk, the walk fails sooner or later. Frost probably has something to do with this, but the trouble occurs in localities where frost is not a factor. If we proceed with the idea that a bond is not required, I think we are inviting a tremendous lot of trouble. Mr. Boynton.

MR. BRETT.—I put a finish on top of a cinder base, 12,000 or 14,000 sq. ft. The coefficient of expansion of that top finish, probably because of free lime in the cement, was very high. The top finish expanded so that it buckled all across the floor. The bond, however, was so good that it pulled up part of the bottom slab. Examination showed that $\frac{1}{2}$ or $\frac{1}{4}$ in. of the bottom cinder concrete was bonded to the top finish. If the top finish had been cut off into squares and laid on an asphalt filler it would not have come up at all. Mr. Brett.

MR. TALBOT.—I believe what Mr. Boynton says concerning the necessity for bond. In our experiments and in our inspection, we find that the floors which are badly cracked or disintegrated on the surface invariably show lack of bond between the top and the structural slab, and the floors which are proving satisfactory show a satisfactory bond. Under heavy loads such as come on a floor in a warehouse where trucks carrying from 500 to 1000 lb. with iron wheels are used, it is absolutely necessary to obtain a bond between the base and the top, or to make the top of sufficient thickness to withstand the traffic independent of the base. Mr. Talbot.

MR. A. J. STAUFFER.—In my experience I have found it necessary to have joints in floors. They are not desired by the owner and sometimes their use is prohibited, but always with resultant failure. Mr. Stauffer.

Mr. Boynton.

MR. BOYNTON.—The objection to joints in floors applies more especially, I presume, to warehouse floors, where there is trucking. I think the joints might be reduced in number in many floors by using reinforcing metal near the top to distribute the stresses that may develop. At the same time I think it would be difficult, if not impossible, to lay a floor over a large area and not have cracks in it if joints are not provided. It might be well to protect the joints in floors in the same manner as that provided for protection of joints in street pavements, by the use of metal angles or plates. It is known that when the joints are made without rounding the edges, they will chip off very rapidly and if the edges are rounded it simply makes it possible for the pounding to begin a little earlier. No doubt you have all noticed in communities where roller skating is practiced the joints pound down very fast. In view of this fact it is not surprising that they break down under heavy trucking. Joints must be eliminated, reduced in number or protected. Our specifications provide for protecting the joints.

AN IMPROVED CONCRETE PAVEMENT.

By E. W. Groves.*

Some years ago the writer became convinced that the forms of street pavement commonly used were too expensive to come into general use, especially on residence streets in both our large and small cities and on country highways. If the demands of modern traffic were to be satisfied, a much cheaper form of pavement must be devised, yet one having all the essential characteristics of permanency and low cost of maintenance. With this idea in view, concrete pavements, constructed in the ordinary manner, were inspected and it was found that while the concrete had the strength to support the weight imposed upon it, it was readily susceptible to the abrasive action of traffic as well as to changes in volume which produced cracks, which in turn, under the action of wheels and hoofs, soon became enlarged into ruts.

Previous to this inspection the writer had caused a section of a street paved with asphalt blocks, which was rapidly disintegrating, to be covered with a thin coating of crude coal tar and sand. The effect was wonderful. The new coating took all of the wear which had formerly been on the asphalt blocks and preserved the pavement, which is in good condition today. The pavement is less noisy than before and not as slippery as sheet asphalt.

For some time the citizens living on a certain block in this city had desired that it be paved. But they were unwilling to pay such high prices as had been charged for pavement. In an effort to satisfy their request the writer conceived the idea of laying a concrete pavement and covering it with a coating of tar and sand. Accordingly, with the consent of the city authorities and the property owners, one block of 1,883 sq. yd. was paved with concrete, covering the surface with hot crude coal tar and sand, using about ½ a gallon of tar to each square yard of pavement. After this pavement had passed through the winter of 1909–1910, and the citizens had an opportunity to judge its merits, petitions

* City Engineer, Ann Arbor, Mich.

were presented asking for over 20,000 sq. yd. which were laid during the summer of 1910. During the winter and spring of 1910 and 1911 petitions were presented asking for about 70,000 sq. yd., of which 64,000 sq. yd. were laid, and at the present time there are on file petitions asking for 140,000 sq. yd. more which will be laid as soon as conditions are favorable.

The writer, of course, was aware that a change of volume of concrete occurs. In a paper before the American Society for Testing Materials, in 1911, A. H. White, of the University of Michigan, gave the results of a series of experiments. In his introduction he speaks of the "Constancy of volume" of cement mortars and calls it a "misnomer." Any one can see from cracked walls, sidewalks and other concrete structures, whose failure is not due to the settling of foundations, that there is some inherent characteristic which causes cement, cement mortars and concrete to change volume if exposed to the weather. Mr. White states:

> The change of volume of concrete due to changes in temperature has been determined with considerable accuracy to be per unit length 0.0000055* or 0.00055 per cent. per deg. Fahr. There are, however, other changes due to the chemical process of setting and hardening which are barely mentioned in even the more important treatises and other variations due to the wetting and drying of the concrete whose very existence is practically unknown.

He deals with these latter and previously unknown changes. From earlier experiments by Schuman, Gary, Considere, and Campbell and White it was definitely determined that neat cements, hardening under water, expand at a decreasing rate for years, and that neat cements, hardening in air, contract similarly, and that cement-sand mortars act in the same manner as neat cement but to a less degree.

Mr. White has been experimenting for 12 years or so with neat cements and various sand mortars. He has found that neat cement when immersed in water for a period of 3 years showed a linear expansion of from 0.10 to 0.15 per cent, nearly all of which occurred during the first year. When these bars were allowed to dry in the room (temperature about 70 deg. Fahr.) for 65 days they had contracted to their original volume and in some cases to 0.05 per cent less. Upon re-immersion in water they regained all their previous expansion in about one day and some expanded

* Pence, *Engineering News*, Vol. 46, 1901, p. 380.

more than they had ever done before. In one specimen of natural rock cement the expansion in water during the same period of time and under the same conditions went as high as 0.55 per cent, but on drying contracted to 0.25 per cent. He also tested in the same manner various bars composed of a 1 : 3 mixture of sand mortar and compound bars of neat cement and the 1 : 3 mixture. The first bars behaved like the neat cement only in a less marked degree. In the compound bars the neat cement part acted in the same manner and in the same degree as the other neat cement bars, while in the 1 : 3 mixture there was a tendency to lag in both expansion and contraction. This difference of expansion of course set up shearing stresses at or near the line of demarcation. In some cases these shearing stresses were great enough to rupture the bars.

To take some examples of this from actual practice, the writer can cite numerous cases in Ann Arbor where the sidewalks have broken apart, that is, the top course broken loose from the bottom course and in every instance it could be seen that the line of fracture did not conform to the line of demarcation but was below in the leaner mixture. There was a film of the base concrete from $\frac{1}{64}$ to $\frac{1}{8}$ in. and in some cases $\frac{1}{2}$ in. in thickness clinging to the top course, showing that the failure was not due to poor bonding between the two courses. Doubtless every one has seen a sidewalk buckle up in an inverted V. This occurred in some of the walks on the campus of the University of Michigan and on several of the city streets in walks that have been laid from 1 to 10 years. The peculiar thing about the failure was that it occurred during the summer months thus precluding any after result of frost action. There remains only the moisture to blame since the expansion due to heat is so small. To the writer's mind the foregoing proves conclusively that the great destruction of concrete pavements is due, first and foremost, to the alternate wetting and drying when they are unprotected.

The coating of bitumen and sand is waterproof. Thus the greatest cause of the destruction of concrete pavement is eliminated. The writer does not care to go into the question of water from below or sub-drainage. The greater portion of the streets of the City of Ann Arbor are of a gravelly soil and thus drain themselves. However a portion of one street is clay and the

pavement was laid directly upon it and shows no ill effect up to the present time, and none is expected.

The pavements laid during the seasons of 1909 and 1910 were treated with crude coal tar and, while this material was fairly satisfactory, it was evident that a distilled product having a uniform consistency would be better. Experiments were made with various products of the distilled tar, tarvia and asphalts with various degrees of viscosity. These were all more or less unsatisfactory, more especially the products containing asphalt oils. As objectionable features made their appearance efforts were made to overcome them. Extensive laboratory tests produced a bitumen which has none of the objectionable features of the crude tar, tarvia and asphalt. This bitumen when applied to the concrete at a temperature of 180 deg. Fahr. is thin enough to bond into the interstices of the base, making an impervious surface.

The pavements laid up to the present time have been made up of two layers of concrete, viz., a base 4½ in. thick composed of 1 part of cement to 8 parts of bank run gravel, and a top layer 1½ in. thick, composed of 1 part of cement to 2 parts of gravel passed through a screen having a 1-in. mesh, the top course being applied immediately after the bottom course is laid to insure a good bond. Since the top coat of a pavement laid in this manner contains more cement than the bottom, and since it has been shown that such a construction tends to produce internal shearing stresses, there is a possibility of separation of the two layers. While no action of this nature is apparent in our pavements, due to the waterproofing effect of the bitumen, to be on the safe side, all our future work will be one coat work, using approximately 1 part of cement to 4½ parts of bank run gravel.

It has been the practice to leave an expansion joint about ¾ of an inch wide at each curb and also every 25 ft. transversely of the street. After the surface of the concrete has been trowled to conform to the crown of the street, and when it is partially set, a light wire broom is used to roughen it in order to furnish a better surface for the adhesion of the bitumen. Later the expansion joints are filled to a little less than 1 in. of the surface with sand upon which is poured hot bitumen, making the joint filling flush with the surface of the concrete. After the concrete has

become thoroughly set and is dry and clean (being swept if necessary) $\frac{1}{2}$ gallon of bitumen, heated to a temperature of 180 deg. Fahr., is spread over each square yard of the surface of the pavement, and while this bitumen is still hot and soft it is covered with a layer of torpedo sand ranging from $\frac{1}{8}$ to $\frac{1}{4}$ in., using about 1 cu. yd. to each 250 sq. yd. of pavement. The street is then immediately opened to traffic which grinds the sand into the bitumen and produces a rubbery surface, black in color, almost noiseless and having the appearance of sheet asphalt.

As to the cost of maintenance, I can only speak from an experience of 3 years. With upwards of 80,000 sq. yd. of this pavement, constructed during the last 3 years, not to exceed $30.00 has been expended on repairs, and the pavement is in good condition today. The owner of a frame house in building does not consider that the first painting will last the life of the house. He plans for the expense of repainting every few years. Its initial cost is low enough for maintenance charges. It must be kept in mind that there is no pavement that is entirely free from maintenance charges.

In summing up the advantages of this pavement, it may be said that the concrete, if properly and honestly constructed, is strong enough to sustain the weight of the traffic which passes over it. The bitumen prevents the absorption of water by the concrete and thus preserves it from cracking. The change of volume due to temperature is negligible. The combination of torpedo sand and bitumen gives a wearing surface which is unexcelled. The construction cost is low as compared with other pavements and requires no cumbersome and extensive plant. When it becomes necessary to put pipes of any sort beneath the pavement repairs are easily and perfectly made, no ridges or hollows being produced. With a firm sub-base no reinforcement whatever is necessary for the stability of the pavement. With the manner of filling the expansion joints described above there is no chipping or crushing of the concrete at the edges. If the filling is carefully done it is impossible to point out an expansion joint in the finished street. When the hot sun of summer heats the pavement there is absolutely no free bitumen coming through and sticking to the feet. It does not "bleed." Our citizens are so well satisfied with it that they will not consider any other

form of pavement. It is, indeed, a most suitable pavement for country highways, parks and boulevards as well as city streets. With gravel banks, ledges of rock, marl, clay and sand, the essential elements of concrete, so profusely distributed throughout this country; the bitumen easily obtained; a pavement that can be laid for about one dollar a square yard, based on Ann Arbor prices, is durable, noiseless, and pleasing to the eye. I can see no reason why it should not become the most universally used of any or all of the various pavements before the people today.

CEMENT PAVING AS CONSTRUCTED AT MASON CITY, IOWA.

By F. P. Wilson.*

For the past five or six years the writer has made a broad study of cement paving and has visited a large number of cities in the Northern States where this class of pavement has been laid and has watched the methods of construction in detail for this class of pavements. A Portland cement concrete pavement, properly laid in an up-to-date manner with first-class cement, good, clean, sharp sand, and good, clean, hard stone, with proper allowance made for expansion and contraction, certainly warrants the use of the same on account of its first cost, cheapness to maintain, the cleanliness of the streets, and the small expense for repair when it becomes necessary to cut holes.

During the summer of 1909 the city of Mason City, Iowa, laid 6,000 sq. yd. of concrete paving in the down-town district, where it would have a test under the most severe traffic. After standing the severe test of two winters and heavy traffic, also the cold of this winter to date, it is at this time in as good a condition as the day it was finished.

During the year 1910 22,000 sq. yd. were laid. The past season 45,000 sq. yd. were laid, and at this writing contracts have been let for 50,000 sq. yd. which this city expects to have laid this coming summer. A total of 70,000 sq. yd. have been laid in Mason City, Iowa, to date at a total cost of $114,378.96, including the cement curbings.

In constructing a first-class concrete pavement, the first requirement is to have strictly first-class material, secondly, to have a first-class, up-to-date set of plans and specifications, and, lastly, a rigid and close following of these specifications in every detail.

The following are the detail specifications for Portland cement

* City Engineer, Mason City, Ia.

concrete pavement used in the construction of all concrete paving laid in Mason City, Iowa, which has proven entirely satisfactory:

PREPARATION OF ROADBED.

All streets, prior to laying any pavement thereon, shall be graded that the pavements will be at the established grade when completed. After excavating to sub-grade, unless the engineer deem the natural ground a proper foundation, excavation shall be continued until solid ground is reached and then refilled to sub-grade with sand, gravel, or broken stone.

The contractor shall be required to remove at his own expense all obstructions, such as trees, old blocks, débris, etc.

EXCAVATION.

All excavated material, gutter stones, planks, macadam, crossing-stones, old curbs, surplus earth, etc., shall be the property of the city and be deposited by the contractor in such place and manner as shall be directed by the engineer the distance not to exceed 3,000 ft. No plowing will be allowed within 3 in. of the bottom of the foundation.

ROLLING.

When the street shall have been graded and shaped to its proper form, it shall be thoroughly rolled with a ten-ton roller to a thoroughly compact surface. If the ground is wet, sand or gravel is to be put in before rolling.

Any depression discovered after this rolling shall be re-filled to sub-grade, re-rolled, and this repeated until a roadbed, perfect as to grade and form, shall have been made.

TAMPING.

When the use of the roller is impracticable, the foundation must be thoroughly puddled and rammed until compacted to the satisfaction of the engineer.

CONCRETE FOUNDATION.

Upon the roadway thus formed will be laid Portland cement concrete five inches thick, to be made as follows: one part by measure of Portland cement; two parts by measure of clean, sharp sand, and five parts by measure of broken stone.

The sand and cement shall be thoroughly mixed into mortar at the proper consistency with a batch mixer approved by the engineer. Broken stone, thoroughly cleaned of dirt, drenched with water, but containing no loose water in the heap, shall then be added to the mortar in the proper proportion. The concrete will then be turned and mixed until each fragment is thoroughly coated with mortar, a strictly wet mixture being required. The concrete thus mixed shall have such a consistency that when rammed the mass will not

shake like jelly, but will, when struck, compact within the area of the face of the hammer without displacing the material latterly.

The concrete thus prepared shall be placed immediately in the work. It shall be spread and thoroughly compacted until free water appears on the surface, which shall be made smooth and parallel to the surface of the finished pavement. The whole operation of mixing and laying each batch of concrete shall be performed in an expeditious and workmanlike manner and be entirely completed before the cement has begun to set.

No retempering of concrete will be permitted, and concrete in which the mortar has begun to set will be rejected.

The thickness of this concrete to be five inches after the same has been compacted.

Extreme care should be taken that the sub-grade is kept moist while this concrete is being put in place.

No concrete shall be laid when the temperature at any time during the day or night falls below thirty-five degrees above zero Fahrenheit.

WEARING SURFACE.

Upon the concrete heretofore specified shall be immediately laid a wearing surface two inches in thickness to be made as follows: One part by measure of Portland cement, two parts by measure of coarse, clean, sharp sand; the sand and cement shall be thoroughly mixed into mortar of the proper consistency with a batch mixer approved by the engineer.

The mortar thus mixed will be immediately laid upon the concrete heretofore specified.

Before this mortar has begun to set it will be finished off to a smooth surface with a wood float, and, before completely hardened, it shall be roughened by brushing with a stiff vegetable brush or broom.

The curvature and cross sections of the pavement to be made according to the plans governing the same.

REQUIREMENTS OF MATERIALS.

The cement used in the work will be submitted to the tests approved and recommended by the American Society of Civil Engineers which it must stand to the satisfaction of the engineer.

All Portland cement used in the work shall be —— —— Portland cement or other Portland cement equally as good, which shall be protected from the weather, free from exposure to air slaking and from moisture until used.

The sand shall be clean, sharp sand.

The stone used for the concrete shall be of the best quality of hard limestone, or other stone equally as good, and shall be broken to such a size that the fragments shall not be larger than will pass through a one and one-half inch ring and not smaller than a hazel nut. It shall be free from dust, dirt, loam or other objectionable material and shall be screened, when necessary, over a one-half inch screen to eliminate dust and small particles.

EXPANSION JOINTS.

An expansion joint one inch in width shall be left next to the curb on each side of the street or alley, also an expansion joint one-half inch in width will be left every twenty-five feet across said pavement at right angles to the curbs. Said expansion joints are to be filled with an asphalt paving filler of proper quality and consistency approved by the engineer. It will be applied while heated to a temperature of about four hundred degrees Fahrenheit, and shall be so applied that said expansion joints shall be thoroughly filled clear to the top of the surface of said pavement.

All forms for expansion and contraction joints shall be made of iron or steel in the form of a template, cut to the desired shape of the street according to the plans, and of sufficient strength to resist springing out of shape. All mortar and dirt shall be removed from forms that have been previously used. The forms shall be well staked to the established lines and grades.

CONTRACTION JOINTS.

Contraction joints shall be made entirely through the pavement every twelve and one-half feet at right angles with the street.

The edges of all expansion and contraction joints shall be rounded to a radius of about one-half inch with proper tools.

Care shall be taken to obtain a surface free from ridges, at expansion or contraction joints, and depressions or unevenness in the surface, that will detract from its appearance, or cause water to lay on the pavement.

Any section having such inferior surface will be rejected and shall be rebuilt by the contractor at his own expense.

Care shall be taken to make the expansion joints in such a manner that they are practically the same width throughout their depth.

Extreme care must be exercised in removing templates or divisions used to make expansion or contraction joints; the breaking out of any portion of the pavement in removing such templates and forms will not be tolerated and such damaged portions of the work shall be torn out and replaced in good condition by the contractor at his expense.

The contractor shall keep pavement sprinkled for one week after it is laid, or longer if deemed necessary by the engineer.

The contractor shall keep the streets barricaded where pavement has been laid at least two weeks after the completion of the same.

The above specification was followed very closely in every detail.

In this work thirty thousand barrels of Portland cement were used. From every car ten samples were taken, tests were made for fineness, tensile strength and specific gravity; also boiling tests were made. In the construction of this work a mechanical batch mixer with a 25-ft. boom, with a traveler on the same, was used.

After the sub-grade had been thoroughly rolled the material was distributed along the street, the rock on one side and the sand on the other. The mechanical mixer was set up at the end of the street 25 ft. from the place of beginning. In the first section, 12½ ft. by 30 ft., the concrete was placed. Immediately the wearing surface was placed upon the concrete, not to exceed 20 minutes elapsing between the time the concrete was placed and the wearing surface was put on the same. Then the next section of 12½ ft. by 30 ft. was put in. Then the mixer propelled itself backwards 25 ft. and proceeded as before.

Parallel with the curb and 10 ft. out from the same the wearing surface was cut through into the concrete, these parallel cuts being 10 ft. apart so that the actual blocks of concrete are only 12½ by 10 ft.

Some of the streets where the cement paving was laid was very soft and swampy. To obtain a dry and well-drained sub-grade, a trench parallel with the curb on each side of the street and 18 in. out from the curb and 18 in. below sub-grade was excavated and a 4-in. drain tile laid in the same and said drain tile connected with the sewers. The earth excavated from said trenches was hauled away and said trenches were re-filled with good, clean, hard, burned cinders, making a thorough drainage for the sub-grade.

The contract price for these cement pavements, including excavations, was $1.32 per sq. yd.

The cost to property owner was 5 cts. per sq. yd. in addition to the contract price, which includes the cost of engineering, inspection, advertising and levying the assessment, making a total cost of $1.37 per sq. yd. to the lot owners abutting on said pavement.

The specifications for the work that is to be constructed this coming season, call for expansion joints to be placed 37½ ft. apart, at right angles to the curb, instead of 25 ft. apart. A soft steel reinforcement plate, ¼ in. in thickness and 2 in. in depth, is required to be anchored back into the wearing surface; said steel plates to be used on each side of the expansion joint to protect the edges of the pavement at that point. The reason for using these expansion protection plates is that the weakest point of a concrete pavement is at the expansion joints.

DISCUSSION.

Mr. Ash.

MR. L. R. ASH.—It seems to me that too much is made of expansion joints in concrete pavements. One street in Kansas City, 1200 ft. long and 36 ft. wide, is without an expansion joint in it, and it was laid under very unfavorable conditions for concrete paving, in that freezing weather occurred during the time it was being laid and very cold weather set in a short time after it was finished. The only cracks ever noticed in this pavement were at intervals of approximately 125 ft., almost as regular as though measured off with a tape line. The cracks did not exceed $\frac{1}{8}$ in. in width. We never have trouble at all in alleys where no expansion joints are placed.

I never heard that concrete had a greater volume than when first laid. The fact that its change in volume is a diminution rather than an increase, simply seems to me to indicate that in any concrete pavement there should be placed lines of weakness occasionally, which can be very readily done by finishing the pavement at regular intervals against a vertical plank across the street. I certainly would not want a longitudinal joint in the pavement.

As for the two course pavement, I do not think it is to be considered in any sense as competing with the one course type, because of the increased cost. With a little care in finishing the pavement and mixing the aggregate a wearing surface can be secured that will be practically the equal of the two course pavement. Some two course pavement was tried when first laying concrete in Kansas City, and was discontinued entirely.

Treating the surface of a concrete pavement with an asphaltic or tar coat, will solve I think, the problem of a cheap form of pavement for moderate traffic better than it has been to date.

Another thing that we have done here with success, and would do more if our means of collecting tax bills were not so complicated, is to lay the pavement the full width, including the distance over the gutter, that is, out to out of curb. Immediately after laying the pavement, a curb form is placed on top

of the fresh concrete and the curb molded right on the pavement. That has been very successfully done with a consequent saving of several cents per lineal foot on curb cost. **Mr. Ash.**

Referring to the pavement first mentioned, that is the one laid under the adverse conditions, which is on Sixth Street in Kansas City, from Bluff to Broadway, that carries probably the heaviest traffic of any street in the city. Over 5,000 vehicles, most of them heavily loaded, have been counted passing the street in 24 hours, and of course that means the majority went over it in a 10-hour period. The street was laid with a thickness of concrete of about 6 to 8 in., because the roadway itself was not quite uniform in its elevation. It was laid with the idea of tiding over a period of litigation when we had almost no street at all. Christmas holidays were approaching and the merchants anxious to have the street opened, so we began in the face of approaching cold weather and hurried along as rapidly as possible, without taking the precautions that we would under more favorable conditions. The result was that the west half of the street, which was laid by using a batch mixer, is not good, although the entire street more than met the expectations originally entertained for it. But the east half of the street, which was more uniformly mixed and deposited in a better manner, has surprised me very greatly as to its wear. When the street is free from ice, snow and dirt, with which it is covered now, I have seen it when it looked like a terrazzo surface. It does wear, no doubt about that, but it wears smoothly, without breaking into holes or pits.

It seems to me the action of this street, under the very unfavorable conditions as to traffic and also as to its original construction, demonstrates very conclusively what we may expect from well constructed concrete streets. It was a one-course pavement laid in the proportions of $1 : 2\frac{1}{2} : 4\frac{1}{2}$ with a limestone aggregate. I may also add that because of the very great hurry in making that street, the contractor was allowed to put back into the street some of the broken concrete obtained from the old base.

THE PRESIDENT.—The Chair would like to take exception to the conclusion that Mr. Groves draws from the tests as Professor White did. The conclusion is that the alternate wetting and drying causes an expansion and contraction, and that when **The President.**

The President. the pavement is in two layers of different density the contraction and expansion might cause curvature and that curvature of the two layers would cause cracking. These tests were made on small test pieces about $4\frac{1}{2}$ in. long and $\frac{1}{2}$ in. square and the relation of the mass to the surface is very much greater than in the case of sidewalks or other mass concrete, and it is not fair to draw such conclusions from tests of this character as applied to large masses of concrete.

This conclusion was drawn as to the sidewalks in Ann Arbor. We all know that there are any number of sidewalks, two course work, all over the country that do not show such conditions. And the conclusion must inevitably be that the workmanship in the Ann Arbor pavement was bad. I think it is difficult to positively prove that the destruction of pavements by cracking is due to the alternate wetting and drying. I think the great trouble with concrete pavements is due to the expansion and contraction, which is entirely independent of the alternate wetting and drying. I think Mr. Ash expressed it clearly when he says that he doubts very much whether there is much change of volume of the concrete due to wetting and drying after it has once set.

Mr. Groves. MR. E. W. GROVES.—At a recent meeting of the Ohio State Engineers, a representative of the U. S. office of Public Roads stated they had observed that there is a change of volume due to the alternate wetting and drying of cement, confirming the experiments by Professor White.

The President. THE PRESIDENT.—I do not mean to say that concrete does not expand and contract under alternate wetting and drying. It is a fact, that Professor White has shown by his tests. The point I do want to take exception to is that alternate wetting and drying affects the cracking and breaking up of sidewalks and roadways.

Mr. Ash. MR. ASH.—I would like to ask whether concrete increases in volume after having been deposited over that at the time of placing. It is generally deposited under at least average temperature conditions, and as far as I have been familiar with concrete, it always contracts. I have built a good many retaining walls, arches and other structures of concrete, and always experience contraction and not very much expansion. If any one has observed the contrary I would like to know it.

THE PRESIDENT.—I think there is no question but that concrete, when it is dry, contracts, and when it is wet increases in volume. There is no doubt that concrete does expand and contract under the action of temperature changes. The President.

MR. ALLEN BRETT.—I have a photograph showing where a long stretch of walk expanded and pushed the curb out, which would show that the walk has at times occupied a larger space than when it was laid. Mr. Brett.

MR. ASH.—I have seen that. I have seen it caused by cracks in the sidewalk being filled with something else and then pushed out. You want to look out for what is in the cracks. Mr. Ash.

MR. F. C. DREHER.—A roadway was constructed by the Highway Department of the City and County of Denver by day labor, about 16 ft. wide and $1\frac{1}{4}$ miles long, concrete about 6 in. thick laid in one course (no mortar coat was used) on natural base which required little preparation, crown of about 4 in. The material used was pit run gravel and cement. Mr. Dreher.

No joints were allowed for expansion, but contraction apparently was the cause of the majority of joints that developed shortly after the completion of the work. A thin coating of tar or asphalt was then placed, which today shows wear in a few places.

Many miles of alley paving have been constructed in this city, 6 in. thick with open joints 10 ft. center at right angles to property line.

MR. F. P. WILSON.—A roadway about 2,000 ft. long, 30 ft. wide, without expansion joints, cracked zig-zag, some places 25 ft., some places 50 ft. and some 100 ft. apart. That roadway had a clay sub-soil base. I think with regard to this construction and the expansion of cement pavements, it depends altogether upon the sub-soil that it is laid upon. Mr. Wilson.

MR. WILLIS WHITED.—It is not uncommon in very hot, dry weather in summer, where there is a long strip of sidewalk, to see a slab buckle up. I have seen them come up a foot; that certainly is not due to moisture. Mr. Whited.

THE PRESIDENT.—Sidewalks shrink in winter, the joints pull apart, the heat of summer expands the concrete and the material that gets in the joints causes the raising. The President.

Mr. Arp.

MR. C. K. ARP.—Mr. Groves mentioned the fact that he used a proportion of 1 : 8. I would like to know whether he used an ordinary bank run of gravel.

Mr. Groves.

MR. GROVES.—We have had no arbitrary accurate quantities of gravel. The coarse aggregate is not in exactly the correct proportion in all cases, but is used as it comes from the bank. It generally has sufficient coarse material to make satisfactory concrete. I am sure if anybody was very critical they could find a good deal of fault with our mixture. The fact remains, however, we have the pavement. I am putting in a good quality of cement, averaging last year 0.94 of a bag to every square yard of pavement.

A proportion of 1 : 8 means 1 part of cement to 8 parts gravel as it comes from the bank. This may not be theoretically correct, but if the gravel is screened, separated and washed, and then put back together again, I cannot build a pavement for 75 or 76 cts. per sq. yd. Of course if the material is separated and put together again, it would make a better concrete; but I make a concrete that is good enough for the purpose. Some have built concrete streets, going many miles for broken stone; it cost a dollar a yard and the people stopped it. With our pavement the traffic of wagon wheels and horses never comes in contact with the road; all that is needed is a good concrete to support traffic and this coating on top takes up the wear.

Mr. Wilson.

MR. WILSON.—Did I understand Mr. Groves to say that he used expansion joints?

Mr. Groves.

MR. GROVES.—Yes, sir; I used expansion joints. The first I used were 20 ft. apart and no cracks appeared, and now I put joints 25 ft. apart, some of them up as high as 35 ft. I have been very strongly urged to leave them out entirely, by people who thought they knew; but we have had such good success with our pavement I do not feel like doing that. I have a great respect for Mr. White's ability as a scientist, and he advises me strongly not to leave out these expansion joints.

Mr. Wilson.

MR. WILSON.—I have had experience both ways. It has been my experience that it is unsuccessful to leave out the expansion joints altogether. I have built several miles of pavement both ways and was unsuccessful one way and successful the other. So I have decided to take the successful way and always put in an expansion joint.

REPORT OF COMMITTEE ON BUILDING BLOCKS AND CEMENT PRODUCTS

The Committee begs to report that after careful study and discussion of the existing specifications and the several subjects which come under its jurisdiction, it has been deemed advisable to make more or less radical changes and revisions in order to meet the requirements which steady progress has made in the last few years.

There are offered proposed specifications for Concrete Architectural Stone, Building Blocks and Brick,* which embody simply the test requirements and methods of testing concrete building products. These specifications are accompanied with Regulations for their use in building construction,* and also a Recommended Practice for their manufacture.* In general these specifications, regulations and recommended practice include all the headings now to be found in Standard No. 3, Standard Specifications for Hollow Cement Building Blocks, with the exception of such revisions as deemed necessary in order to make them applicable to modern practice.

The Committee also presents a proposed Recommended Practice for the Manufacture of Plain Concrete Drain Tile.* Inasmuch as standard test requirements for this class of concrete products are yet to be established, the Committee hesitates to submit specifications covering same.

There is at least one type of concrete product, namely: Concrete Fence Posts, which this Committee would like to bring before the Association at this time for general discussion with the earnest hope of obtaining data which will lead to the general test requirements, etc., which may eventually result in a specification and recommended practice for their manufacture and use.

Respectfully submitted,

PERCY S. HUDSON, *Chairman.*
CLARENCE K. ARP,
P. H. ATWOOD,
ROBERT F. HAVLIK,
CHARLES D. WATSON.

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* The specifications appear in the following pages as amended and adopted by letter ballot May 15, 1912.—ED.

NATIONAL ASSOCIATION OF CEMENT USERS.

PHILADELPHIA, PA.

RECOMMENDED PRACTICE FOR PLAIN CONCRETE DRAIN TILE.

ADOPTED, MAY 15, 1912.

This recommended practice is intended to cover the general requirements for the manufacture of plain concrete drain tile.

MATERIALS.

Cement. 1. The cement shall meet the requirements of the Standard Specifications for Portland Cement of the American Society for Testing Materials, and adopted by this Association. (Standard No. 1.)

Aggregates. 2. The aggregates shall be clean, coarse, hard, durable materials, and shall be free from dust, soft, flat or elongated particles, loam, vegetable or other deleterious matter. In no case shall aggregate containing frost or lumps of frozen material be used.

(a) *Fine Aggregate.*—The fine aggregate shall consist of sand, crushed stone or gravel screenings, preferably of silicious material, graded from fine to coarse, and passing, when dry, a screen having one-quarter ($\frac{1}{4}$) inch diameter holes; not more than twenty (20) per cent. shall pass a sieve having fifty (50) meshes per linear inch, and not more than six (6) per cent. pass a sieve having one hundred (100) meshes per linear inch.

Fine aggregate shall be of such quality that mortar composed of one (1) part Portland cement and three (3) parts fine aggregate by weight when made into briquettes will show a tensile strength at least equal to 1:3 mortar of the same consistency made with the same cement and Standard Ottawa sand.

(b) *Coarse Aggregate.*—The coarse aggregate shall consist of gravel, crushed stone or other suitable material graded in size,

which is retained on a screen having one-quarter (¼) inch diameter holes.

3. Water shall be clean, free from oil, acid, strong alkalies or vegetable matter. **Water.**

PROPORTIONS.

4. The proportions of cement to aggregate shall be such as to require at least the minimum amount of cement to produce strength and density as hereinafter specified. The proportions of the various sizes of aggregates and cement to aggregates shall preferably be made by weight. If by volume, a bag of Portland cement shall be considered 1 cubic foot. **Proportions.**

5. Methods of measurement of the proportions of the various ingredients shall be used which will secure uniform measurements at all times.

MIXING.

6. The ingredients of concrete shall be thoroughly mixed dry, sufficient water added to obtain the desired consistency, and the mixing shall continue until the cement is uniformly distributed and the mass is uniform in color and homogeneous. The mixing shall preferably be done with a machine mixer of a type which insures the proper mixing of the materials throughout the mass. **Mixing.**

7. (*a*) *Semi-wet Process.*—The consistency of the concrete shall be such as to show web-like markings on the surface of the tile when the forms are removed. **Consistency.**

(*b*) *Wet Process.*—The consistency of the concrete shall be such that it will be forced into every part of the mold by jarring or tapping.

8. Retempering or using mortar or concrete 40 minutes after being mixed shall not be permitted. **Retempering.**

CURING.

9. *Natural Curing.*—The tile shall be protected from the sun and strong currents of air for a period of at least 7 days. During this period they shall be sprinkled at such intervals as is necessary to prevent drying, and maintained at a temperature of not less than 50° F. Such other precautions shall be taken as to enable the hardening to take place under the most favorable conditions. **Curing.**

After 7 days, the tile may be removed to the yard but in no case used before they are 28 days old.

10. *Steam Curing.*—The tile shall be removed from the molds as soon as conditions will permit and shall be placed in an atmosphere of steam saturated with moisture for a period of at least 48 hours. The tile shall then be removed and stored for at least 8 days. The steam-curing chamber shall contain an atmosphere saturated with moisture and maintained at a temperature between 100° and 130° F. (This does not apply to pressure steam curing.)

DIMENSIONS.

Dimensions. 11. (*a*) *Diameter.*—The diameter or size of the tile shall refer to the inside diameter and be uniform in all directions.

(*b*) *Thickness.*—From 4 to and including 22 in. in diameter, the wall thickness shall not be less than one-twelfth of the diameter. Tile above 22 in. in diameter shall have a wall thickness of not less than one-tenth of the diameter.

(*c*) *Length.*—The length of the tile shall be uniform at all points, and preferably not less than the diameter, with a minimum of 12 in.

STANDARD No. 10.

NATIONAL ASSOCIATION OF CEMENT USERS.

PHILADELPHIA, PA.

RECOMMENDED PRACTICE FOR CONCRETE ARCHITECTURAL STONE, BUILDING BLOCK AND BRICK.

ADOPTED MAY 15, 1912.

GENERAL.

1. This Recommended Practice is intended to cover the general requirements for the manufacture and testing of concrete architectural stone, building blocks and brick.

MATERIALS.

Cement. 2. The cement shall meet the requirements of the Standard Specifications for Portland cement of the American Society for Testing Materials, and adopted by this Association. (Standard No. 1.)

Aggregates. 3. The aggregates shall be clean, coarse, hard, durable materials, and shall be free from dust, soft, flat or elongated particles, loam, vegetable or other deleterious matter. In no case shall aggregate containing frost or lumps of frozen material be used.

(a) *Fine Aggregate.*—The fine aggregate shall consist of sand, crushed stone or gravel screenings, preferably of silicious material, graded from fine to coarse, and passing, when dry, a screen having one-quarter ($\frac{1}{4}$) inch diameter holes; not more than twenty (20) per cent. shall pass a sieve having fifty (50) meshes per linear inch, and not more than six (6) per cent. pass a sieve having one hundred (100) meshes per linear inch.

Fine aggregate shall be of such quality that mortar composed of one (1) part Portland cement and three (3) parts fine aggregate by weight when made into briquettes will show a tensile strength at least equal to the strength of 1:3 mortar of the same consistency made with the same cement and Standard Ottawa sand.

(b) *Coarse Aggregate.*—The coarse aggregate shall consist of gravel, crushed stone or other suitable material graded in size, which is retained on a screen having one-quarter ($\frac{1}{4}$) inch diameter holes. In no case shall the maximum dimension be greater than one-half the minimum width of any section of the finished product.

Coloring Matter. 4. Where color is required, only the most permanent and durable mineral colors shall be used and shall be considered as aggregate in measuring proportions.

Water. 5. The water shall be clean, free from oil, acid, strong alkalies or vegetable matter.

PROPORTIONS.

Proportions. 6. The proportions of cement to aggregate shall be such as require at least the minimum amount of cement to produce the strength and density specified in the Standard Specifications for Concrete Architectural Stone, Building Blocks and Brick. The proportions of the various sizes of aggregates and cement to aggregates shall preferably be made by weight. If by volume, a bag of Portland cement shall be considered one (1) cubic foot.

7. *Measuring Proportions.*—Methods of measurement of the proportions of the various ingredients shall be used which will secure uniform measurements at all times.

MIXING.

Mixing. 8. The ingredients of concrete shall be thoroughly mixed dry, sufficient water added to obtain the desired consistency, and the mixing shall continue until the cement[1] is uniformly distributed and the mass is uniform in color and homogeneous. The mixing shall preferably be done with a machine mixer of a type which insures the proper mixing of the materials throughout the mass.

Consistency. 9. (a) *Wet Process.*—The concrete must have at least a sufficient amount of water to make it so soft that it must be handled quickly to prevent it running off the shovel, but not so thin as to cause segregation of the materials.

(b) *Semi-wet Process.*—The material shall be mixed with a maximum amount of water permissible, and must have sufficient

water so that the mixture will hold its form after being compressed in the hand.

10. Retempering, that is remixing mortar or concrete partially hardened with additional water, or using mortar or concrete forty minutes after being mixed, shall not be permitted. **Retempering.**

REINFORCEMENT.

11. All lintels, bearing stones and other members subjected to cross bending shall be reinforced by means of rods placed about one and one-half inches from their tension surface, and the total sectional area for the reinforcement shall not be less than 0.8 of 1 per cent. of the cross sectional area of the concrete in the member reinforced. When any member exceeds in any dimension eight times its least dimension, it shall be reinforced to insure safety in handling. **Reinforcement.**

CURING.

12. *Natural Curing.*—The concrete products shall be protected from the sun and strong currents of air for a period of at least 7 days. During this period they shall be sprinkled at such intervals as is necessary to prevent drying, and maintained at a temperature of not less than 50° F. Such other precautions shall be taken as to enable the hardening to take place under the most favorable conditions. After 7 days the products may be removed to the yard but in no case used before they are 21 days old. **Curing.**

13. *Steam Curing.*—The products shall be removed from the molds as soon as conditions will permit and shall be placed in a steam-curing chamber containing an atmosphere of steam saturated with moisture for a period of at least 48 hours. The curing chamber shall be maintained at a temperature between 100° and 130° F. The products shall then be removed and stored for at least 8 days. (This does not apply to high pressure steam curing.) **Steam Curing.**

FINISHING, MARKING AND HANDLING.

14. Concrete products may have exposed surfaces treated by any of the various methods proposed by this Association in the Report on Treatment of Concrete Surfaces. All surfaces and arrises of stone must be true and without imperfections. **Finish.**

Marking. 15. All concrete products of full standard size shall be marked for purpose of identification, showing name of manufacture or brand, date (day, month, and year) made.

Handling. 16. All concrete products shall be handled with utmost care. When transported and subjected to rough handling they shall be crated and packed in non-staining material in such a way as to insure no damage from chipping or abrasion. All large and heavy stone shall be provided with hooks for lifting. When necessary stone shall be provided with metal bonds for the purpose of tying to the masonry backing.

STANDARD No. 11.

NATIONAL ASSOCIATION OF CEMENT USERS.

PHILADELPHIA, PA.

STANDARD SPECIFICATIONS FOR CONCRETE ARCHITECTURAL STONE, BUILDING BLOCK AND BRICK.

ADOPTED MAY 15, 1912.

I. TEST REQUIREMENTS.

1. Concrete architectural stone, building block and brick must be subjected to compression and absorption tests. The test samples must represent the ordinary commercial product of the regular size and shape used in construction.

2. (*a*) *Solid concrete stone, block and brick.*—In the case of solid concrete stone, block and brick the ultimate compressive strength at 28 days must average 1,500 lb. per sq. in. of gross cross-sectional area of the stone as used in the wall, and must not fall below 150 lb. per sq. in. in any case. **Compression.**

(*b*) *Hollow and two-piece building blocks.*—The ultimate compressive strength of hollow and two-piece building blocks at 28 days must average 1,000 lb. per sq. in. of gross cross-sectional area of the block as used in the wall, and must not fall below 700 lb. per sq. in. in any case.

(*c*) *Area of hollow blocks.*—In the case of hollow building blocks the gross cross-sectional area shall be considered as the actual wall area including the block and air space displaced by the block.

(*d*) *Area of two-piece blocks.*—In the case of two-piece blocks the blocks shall be tested in pairs consisting of the front and rear blocks as used in the wall. The compressive strength shall be regarded as the sum total sustained by the two blocks divided by the product of the length of the blocks and the width of the wall.

Absorption. 3. The percentage of absorption at 28 days (being the weight of the water absorbed, divided by the weight of the dry sample) must not exceed five (5) per cent. when tested, as hereinafter specified.

II. STANDARD METHODS OF TESTING.

General. 4. (*a*) *Laboratory.*—All tests required for approval shall be made in some laboratory of recognized standing.

(*b*) *Samples.*—For the purpose of the tests at least nine samples or test pieces must be provided. Such samples must represent the ordinary commercial product, and shall be selected from stock. In cases where the material is made and used in special shapes or forms too large for testing in the ordinary machines, smaller size specimens shall be used as may be directed.

(*c*) *Tests.*—Tests shall be made in series of at least three. The remaining samples are kept in reserve in case duplicate or confirmatory tests be required. All samples must be marked for identification and comparison.

Compression Tests. 5. The compression tests shall be made as follows:

(*a*) *Solid concrete stone, block and brick.*—When testing solid concrete stone, block and brick, the net area shall be considered as the minimum area in compression.

(*b*) *Hollow and two-piece building blocks.*—Whenever possible such tests shall be made on full size blocks. When such tests must be made on portions of blocks, both pieces of the block must be tested and both must contain at least one full web section. The samples must be carefully measured, then bedded flatwise in plaster of Paris to secure uniform bearing in the testing machine and crushed.

The net area shall be regarded as the smallest bearing area of the piece being tested. The total compressive strength shall be divided by the net area to obtain the compressive strength in lb. per sq. in. of net area of each piece. The sum of the two results shall then be averaged to obtain the average strength in lb. per sq. in. of the net area of the total block.

The entire block shall be carefully measured to determine the maximum air space prior to breaking the block for the compressing tests, and the net compressive strength obtained shall then

be reduced to compressive strength in lb. per sq. in. of gross area, this being figured from the actual air space of the block determined above.

Absorption Tests.

6. The sample is first thoroughly dried to a constant weight at not to exceed 212° F. and the weight carefully recorded. When dried, the sample is to be immersed in a pan or tray of water to a depth of 2 in. resting on two strips not over 1 in. in width to allow the water to have free access to face. At the end of 48 hours from the time it is placed in water, the sample shall be removed, the surface water wiped off, and the sample carefully weighed.

NATIONAL ASSOCIATION OF CEMENT USERS.

PHILADELPHIA, PA.

STANDARD BUILDING REGULATIONS FOR THE USE OF CONCRETE ARCHITECTURAL STONE, BUILDING BLOCK AND BRICK.

ADOPTED MAY 15, 1912.

I. GENERAL.

Class of Buildings. 1. Concrete Architectural Stone, Building Block and Brick meeting the requirements set forth in the Standard Specifications and Standard Recommended Practice may be used in building construction, subject to the usual form of approval required of other materials of construction by the Bureau of Building Inspection.

Height of Buildings. 2. The height of buildings constructed of concrete building products shall be limited by the requirements in these regulations.

II. DETAILS OF CONSTRUCTION.

Thickness of Walls. 3. (*a*) *Bearing Walls, 25 ft. Span.*—The thickness of bearing walls in such buildings as garages, stables, office buildings, hotels, tenements, boarding and lodging houses and residences shall be as given in the table below, for buildings in which the maximum distance between bearing walls or columns does not exceed 25 feet.

No. of Stories.	THICKNESS OF WALL IN INCHES.				
	Basement.	1st Story.	2d Story.	3d Story.	4th Story.
1	8	8	....	....	....
2	10	8	8	....	....
3	12	10	8	8	....
4	16	12	10	8	8

(*b*) *Bearing Walls, More Than 25 ft. Span.*—In buildings not covered by the above, the thicknesses of the bearing walls shall be

determined according to the limit of loading specified in Paragraph 6. In no case, however, shall any outside bearing wall be less than eight (8) inches thick.

(c) *Party Walls.*—Hollow concrete blocks used in the construction of party walls shall be filled solid with concrete placed on the job.

(d) *Curtain or Partition Walls.*—For curtain walls or partition walls, the requirements shall be the same as in the use of hollow tile, terra cotta or plaster blocks.

Walls, Laying, etc.

4. (a) *Bonding.*—Where the face only is of hollow cement block, and the backing is of brick, the facing of hollow block must be strongly bonded to the brick, either with headers projecting four (4) inches into the brick work, every fourth course being a header course, or with approved ties, no brick backing to be less than eight (8) inches thick. Where the walls are made entirely of concrete blocks, but where said blocks have not the same width as the wall, every fifth course shall extend through the wall, forming a secure bond, when not otherwise sufficiently bonded.

(b) *Portland cement mortar* shall be made of Portland cement and sand in the proportions of one (1) part cement and not more than two and one-half ($2\frac{1}{2}$) parts sand, and shall be used immediately after being mixed.

(c) *Portland cement and lime mortar* shall be made of Portland cement and sand in the proportions by volume of one (1) part cement, not more than two and one-half ($2\frac{1}{2}$) parts sand and not more than one-quarter ($\frac{1}{4}$) part hydrated or thoroughly slaked lime.

Girders or Joists.

5. Wherever girders or joists rest upon walls so that there is a concentrated load on the block of over two (2) tons, the blocks supporting the girder or joists must be made solid for at least eight (8) inches from the inside face. Where such concentrated load exceeds five (5) tons, the blocks for at least three courses below, and for a distance extending at least eighteen (18) inches each side of each girder shall be made solid for at least eight (8) inches from the inside face. Wherever walls are decreased in thickness, the top course of the thicker wall shall afford a full solid bearing for the webs and walls of the course of blocks above.

Limit of Loading.

6. No wall composed of hollow concrete block when laid up in a Portland cement and lime mortar shall be loaded at any point to an excess of 167 lb. per sq. in. equivalent to twelve (12) tons

per square foot of the superficial area of such blocks as used in the wall including the weight of the wall. When the blocks are laid up in a Portland cement mortar, this limit of loading may be increased to 200 lb. per sq. in. In buildings where most of the floor loads, etc., are carried by pilasters, said pilasters may be made of hollow concrete building blocks and the air spaces filled in solid with slush concrete placed on the job. Such pilasters shall not be loaded to exceed 300 lb. per sq. in. of gross cross-sectional area.

Strength of Blocks. 7. No blocks shall be used in bearing walls that have a crushing strength of less than 1,000 lb. per sq. in. of gross cross-sectional area at the age of 28 days.

Hollow Space. 8. The hollow space in building blocks used in bearing walls shall not exceed 33 per cent. except where blocks containing a greater percentage shall be proved by actual tests to meet all the test requirements herein specified to the satisfaction of the Bureau of Building Inspection.

METHOD OF TESTING DRAIN TILE.*

By Arthur N. Talbot† and Duff A. Abrams.‡

Recent developments in the manufacture of farm drain tile have emphasized the importance of having a simple standard portable testing machine which may be used for making tests of drain tile in the field or at the plant. The increasing use of tile of large size in farm drainage districts is well known. The competition between clay tile and the concrete tile has brought up new questions. What strength shall be required for tile of a given size in order that they may be considered to be commercially first-class tile? In the case of concrete tile, what thickness, richness of mixture, method of curing and age at laying are necessary to fill the requirements for a first-class article?

A number of elements enter into the choice of a suitable method of determining the physical properties of drain tile:

(1) A definite and important quality should be determined by the test.

(2) The test should be simple, easily and quickly made and should not require the services of an expert laboratory man.

(3) The test should be of such a character as not to give unduly diverse results for test pieces of the same grade.

(4) The machine to be used should be simple and inexpensive, readily adjusted to different sizes of test pieces and easily transported from point to point and made ready for use.

It is believed that the machine described in this article satisfactorily fulfills the requirements for making field tests of drain tile. This machine was designed by D. A. Abrams for use in the Laboratory of Applied Mechanics of the University of Illinois.

The machine consists essentially of a simple frame-work and a lever for applying the load by means of dead weight. The load

* A part of the material in this paper was presented at the annual meeting of the Interstate Cement Tile Manufacturers' Association at Chicago, February 22, 1911.

† Professor of Municipal and Sanitary Engineering, in charge of Theoretical and Applied Mechanics, University of Illinois.

‡ Associate, Engineering Experiment Station, University of Illinois.

applied through the loading lever may be blocks of iron, stone, sand or other suitable material. After the test the dead load is weighed. To obtain the load on the tile this weight is multiplied by ten and a constant quantity due to the weight of the loading lever (about 100 lb. in this particular machine) is added.

The machine with a 30-in. clay tile in place ready for loading is shown in Fig. 1. Fig. 2 gives the principal dimensions of the different parts. The machine measures 30 in. between uprights and will take tile up to 42 in. inside diameter. The main members

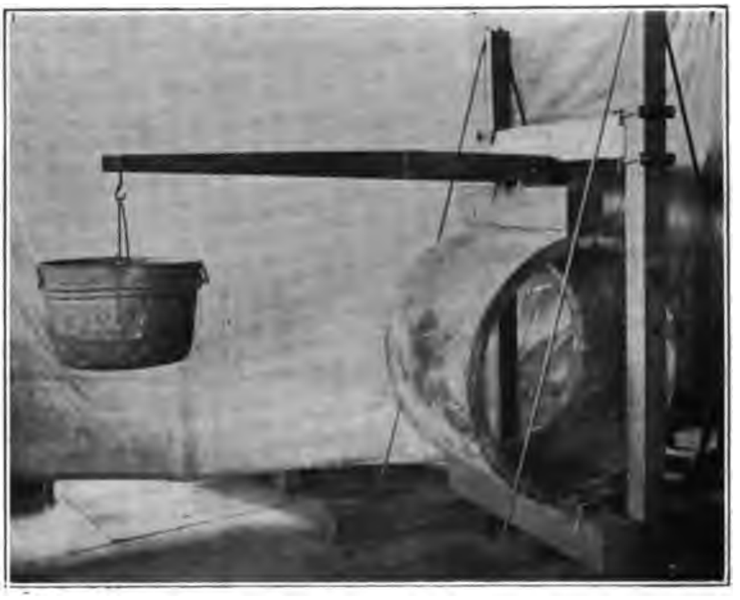

FIG. 1.—TILE UNDER TEST.

are of timber; metal plates and other shapes are used at points of concentrated load and for connections.

Metal knife edges are provided for the bearing of the loading lever on the top loading block and for taking the upward thrust against the top cross block. The knife-edge bearings on the block over the test tile are 5 in. center to center and a single knife edge takes the end thrust. This gives considerable freedom to the top loading block and allows the load to be fairly central, although the top and bottom elements may not be parallel,

The bottom loading block is provided with two small half-rounds of hard wood placed about 2 in. apart, which allow the tile to seat itself in place. The load is applied at the top along a single element. Cushions consisting of short lengths of flattened rubber-lined fire hose serve to distribute the load along the length of the tile and prevent any local concentration of the load due to irregularities in the top or bottom surfaces.

The top cross block can be placed and held in any position along the uprights to accommodate the machine to any diameter of tile up to about 42 in. By this means the machine is adjustable

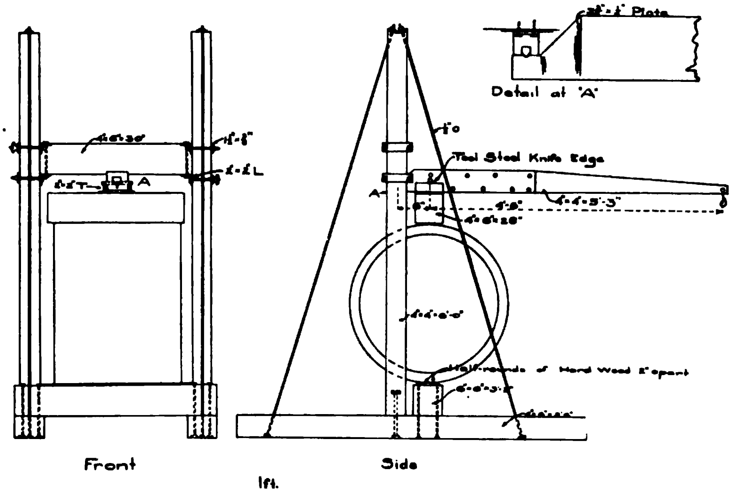

FIG. 2.—DETAILS OF TESTING APPARATUS.

to the greatest variation in the size of the test tile and will apply the load to any sizes under uniform conditions.

In order to check the dimensions of the loading lever, it was calibrated by setting a pair of platform scales in the machine and loading up to about 500 lb. on the machine. It was then placed in a 10,000 lb. testing machine and loaded up to 4,000 lb. The greatest error observed for this range of load was less than 1 per cent.

This machine weighs 225 lb. It should not cost more than $15 to $18 in a shop equipped for wood and metal working.

Up to the present date about one hundred tests have been

made on this machine on concrete and clay tile in sizes 12 to 36 in. inside diameter. The breaking loads varied from 1,400 to 5,000 lb. per tile.

An examination of this testing machine will show that it is simple in operation and that it is easily adjustable for different sizes. Tile which are out-of-round in different ways at the two

TABLE I.—SUMMARY OF TESTS OF CONCRETE DRAIN TILE.

Ref. No.	Average Internal Diameter, in.	Average Thickness, Top and Bottom, in.	Length, in.	Weight, lb.	Age at Test, days.	Maximum Load, lb.	Modulus of Rupture, lb. per sq. in.	Remarks.
36W1	36.7	2.95	23.8	690	177	4,310	770	See note below
36W2	36.0	2.98	24.0	700	177	4,680	819	
36W3	36.0	3.02	23.9	722	177	4,760	812	
						Average	800	
24W1	23.9	2.07	24.0	313	181	2,860	690	
24W2	24.1	2.00	24.0	312	181	1,900	492	
24W3	24.0	1.97	24.0	316	...	2,660	708	
						Average	615	
20W1	20.3	1.74	23.9	205	163	1,770	515	
20W2	20.3	1.63	23.9	209	159	1,640	539	
20W3	20.3	1.70	23.9	209	159	1,670	516	
						Average	527	
18W1	18.3	1.65	23.8	188	135	3,000	882	
18W2	18.3	1.65	23.8	186	135	2,920	859	
18W3	18.3	1.65	23.8	184	135	3,170	892	
						Average	878	
12W1	11.8	.95	12.2	30	146	850	939	
12W2	11.8	.96	12.2	31	146	820	887	
						Average	913	

The 36-in. tile were reinforced with 2¼-in. square twisted bars, placed at middle of thickness of tile. Bars were welded into circular rings. The modulus of rupture for these tests was computed in the same manner as for the other tile, disregarding the reinforcement.

Concrete consisted of 1 part Portland Cement to 3½ parts washed sand and gravel. Concrete machine mixed.

The 12-in. tile were machine made and were placed in steam chamber for 12 hours. The other tile were stored in the open air.

ends will be easily taken by the machine and there is little chance for an unfair distribution of the load. The strip of hose gives some cushioning effect and the load is practically distributed over the whole length in all cases. The method of loading along a line at the top and bottom was selected because of its simplicity. The arrangement of the machine allows a tile to be rolled into

place and to be easily made ready for test. It is believed that the results obtained by different operators will agree quite closely.

If desired, the modulus of rupture of the material may be determined from the bending moment developed and the dimensions of the pipe. For general purposes it will be preferable to report the load per foot of length of pipe for a given size. Possibly for some purposes it may be interesting to divide this load by the diameter of the pipe in inches and thus compare the results per inch of diameter for a pipe one foot long.

TABLE II.—SUMMARY OF TESTS OF CONCRETE DRAIN TILE.

Ref. No.	Average Internal Diameter, in.	Average Thickness, Top and Bottom, in.	Length, in.	Weight, lb.	Age at Test, days.	Maximum Load, lb.	Modulus of Rupture, lb. per sq. in.	Remarks.
36—1	35.9	3.10	24.3	706	101	1,965	314	
36—2	36.0	3.10	24.2	721	72	1,885	304	Wet concrete
36—3	36.0	2.93	24 0	700	105	2,495	451	
						Average	356	
27—1	26.7	2.81	24.3	454	65	2,135	311	
27—2	26.6	2.90	24.6	460	84	2,945	411	
27—3	26.6	2.84	24.5	475	60	2,305	327	
						Average	350	
22—1	22.0	2.24	24.4	321	51	2,665	500	
22—2	22.0	2.23	24.1	335	68	2,885	557	Wet Concrete
22—3	22.0	2.27	24.2	315	..	1,890	350	
						Average	469	
12—1	12.0	1.05	12.3	39.6	..	1,405	1,290	Machine made
12—2	12.0	1.05	12.2	39.3	..	1,295	1,200	do
12—3	12.0	1.05	12.4	39.3	..	1,345	1,225	do
						Average	1,238	

The 36, 27, and 22-in. tile were made at a field plant about two miles west of Champaign, Illinois. Metal forms were used. The forms were removed immediately upon the completion of the tamping.

The 12-in. tile were machine made, having been shipped from the plant at Edinburgh, Ill. These tile had 6 circumferential corrugations about 0.10 in. high and ⅜ in. wide at base.

Details regarding materials used and storage conditions not furnished.

It has seemed the simplest way to fix at a definite distance apart the two strips on which the tile rests. An analysis of rings shows that when the bearings on these strips are 2 in. apart, the formula for the bending moment will be but 2½ per cent different from that for a single support for tile 6 in. in diameter and ½ of 1 per cent. for a tile 12 in. in diameter, while for larger sizes this

variation will be much less. Under the conditions of such tests it would seem better to fix the distance for these strips and use a common expression for the formula for the bending moment for all sizes of tile to be tested. It would seem that 0.16 Qd is a satisfactory expression for the bending moment where Q is the concentrated load applied at the crown and d is the mean diameter of the tile. For the modulus of rupture f of the material the formula would be

$$f = 0.96 \frac{Qd}{lt^2}$$

where l is the length and t the thickness of the tile along the top.

This method of testing was selected in preference to a method involving the bedding of the tile in sand or other material, because of the difficulty in embedding large tile in sand in such a way as to obtain a fair distribution of pressure and in securing the same distribution of pressure in different tests and because the method of concentrated loads will give a more definite index of the strength of the material.

In tests of materials it is not essential that the material shall be subjected to the same action in the process of testing that it will receive in service. The cold bend test of steel is one of the most useful and instructive of tests, but it differs radically from any condition of service in which the steel will be placed. The value of a test will depend upon the properties determined. In testing drain tile the method of applying concentrated loads has many advantages over that of applying distributed loads. Whatever the method of testing used, it will be necessary finally to determine the relation between strength of the test piece and the strength which is needed in the structure. In the case of tile to be used in a ditch of a given depth and a given soil the necessary test strength will have to be determined. Since the tests will have to be translated into the working conditions, it would seem unnecessary to attempt to make the conditions like the conditions in the ditch. It is of much more importance that the tests should be simple, direct and fairly uniform under varying conditions of tile and with different machines and different operators. Experience with this machine leads to the conclusion that it would make a satisfactory means of determining the quality of drain tile.

The three series of tests of drain tile given in Tables I, II and III have been selected as representative of the results obtained with the machine described above and were made at the Laboratory of Applied Mechanics, University of Illinois.

TABLE III.—SUMMARY OF TESTS OF CLAY DRAIN TILE.

Ref. No.	Average Internal Diameter, in.	Average Thickness Top and Bottom, in.	Length, in.	Weight, lb.	Maximum Load, lb.	Modulus of Rupture, lb. per sq. in.	Remarks.
30—1	30.3	2.30	23.8	400	4,850	1,190	Hard burned
30—2	30.5	2.35	24.0	416	3,750	905	Black core
30—3	30.2	2.35	24.0	406	2,830	660	Black core
					Average	915	
27—1	28.0	1.90	25.5	308	3,180	988	
27—2	28.0	1.97	24.5	313	3,640	1,066	
27—3	27.6	1.93	23.5	314	3,840	1,180	
					Average	1,045	
24—1	24.6	1.78	25.5	238	2,360	737	
24—2	24.6	1.71	25.4	241	2,520	818	
24—3	24.5	1.72	25.4	242	3,030	1,055	
					Average	870	
18—1	18.9	1.30	25.8	145	2,040	917	
18—2	19.0	1.32	25.2	144	2,080	915	
18—3	19.0	1.30	25.5	143	2,240	1,025	
					Average	952	
12—1	12.7	1.10	25.8	79.7	1,835	760	
12—2	12.4	1.07	25.2	80.0	2,985	1,318	
12—3	12.6	1.12	25.5	79.3	1,745	713	
					Average	930	

ADVANTAGES AND DURABILITY OF CEMENT SEWER PIPE.

By Gustave Kaufman.*

The matter brought out in this paper is based upon the experience gained from the use of concrete sewer pipe in Brooklyn. As is generally known, about 400 miles of such pipe, below 24 in. in diameter, have been laid and are in use there. It can be truly said that this pipe has given eminent satisfaction to the authorities and has been maintained at a lower cost, per mile, than vitrified clay pipe of the highest grade.

MANUFACTURE.

Before starting upon the real subject of this paper, a description of this pipe, and method of its manufacture, is here given.

Pipe are made in 6-, 9-, 12-, 15-, 18- and 24-in. equivalent capacity. The 6- and 9-in. are plain round pipe. They are all three feet in length, with hub joint, with the exception of the 6-in. which is two and one-quarter feet. The 12-in. pipe is round, with flat base, and the 15-in., 18-in. and 24-in. pipe are egg-shaped, with flat-base. The thickness of the walls is as follows:

Size of Pipe in inches	Thickness of Wall in inches.
6	$\frac{3}{4}$
9	$\frac{7}{8}$
12	$1\frac{1}{16}$
15	$1\frac{3}{8}$
18	$1\frac{5}{8}$
24	2

The collars are as follows:

6-in. collars	.. $1\frac{5}{16}$ in.	in depth, with joint of $\frac{1}{8}$-in.
9-in.	"	.. $1\frac{1}{4}$ in.	in depth, with joint of $\frac{1}{8}$-in.
12-in.	"	.. $1\frac{3}{8}$ in.	in depth ,with joint of $\frac{3}{16}$-in.
15-in.	"	.. $1\frac{5}{8}$ in.	in depth, with joint of $\frac{1}{8}$-in.
18-in.	"	.. $1\frac{5}{8}$ in.	in depth, with joint of $\frac{3}{16}$-in.
24-in.	"	.. $1\frac{3}{8}$ in.	in depth, with joint of $\frac{1}{4}$-in.

* General Manager, Wilson & Baillie Manufacturing Company, Brooklyn, N. Y.

These cement pipe up to 1890 were made by hand, when pipe made by machine were introduced by The Wilson & Baillie Manufacturing Company. With the hand made pipe it was always difficult to thoroughly mix the ingredients of the concrete unless the workmen were closely watched, and frequently resulted in a pipe far from homogeneous and of equal density throughout. In the manufacture of the machine-made pipe the cement, sand and trap rock are measured and thoroughly mixed in the machine mixer, evenly fed to the molds, and rammed by machinery with iron rammers regulated to produce continuous and uniform blows of any impact desired. The result is a product perfectly homogeneous and of equal and great density throughout.

The machine used for manufacturing the pipe consists of a mechanical tamper and a revolving table upon which the molds are placed. The tampers have a vertical reciprocating motion and at the same time move outward and inward rapidly so as to conform to the line of the travel of the mold, which, owing to its oval form, presents, at each revolution, varying diameters to the successive tamping bars. There are 8 tampers, made of the best tool steel, running 200 tamps a minute each. Only one rammer is down at a time. The head, which consists of the actuating machinery for the tampers, is counter-balanced upwards as the mold is being filled with concrete. The head is raised by the density of the concrete and in this way an even and regular product is obtained which it is impossible to achieve by hand. The force of the blow of each rammer is estimated at 800 lb. The area of the arm of the rammer is about 1 in. square.

The proportions, which have been used in later years, are 1½ parts of the best grade of Portland cement, 1 part of sand and 3 parts of trap rock screenings, containing 20 per cent of stone dust. The percentage of water used to the whole bulk varies from 10 to 15 per cent. according to the condition of the ballast. The mixture when dumped on the floor, is apparently dry, but will ball in the hand with some pressure. An extra mixture is used in forming the collar for the reason that as the rammers do not exert a direct blow on the material in the offset compression of the material must be depended upon.

The mixed concrete is delivered to the machines in barrows and is fed into the hoppers by two men, one on either side. As

soon as the flask is full, and the core automatically lifted clear, the flask is taken up by a pipe truck and wheeled into the stripping rooms where it is allowed to stand usually 30 minutes, when it is stripped. After the pipe have set over night a spray of water is turned on and the pipe kept damp until at the expiration of 6 days when they are removed from under cover and placed in the yard. The pipe, at the expiration of 30 days, are set sufficiently to be handled in the work.

Spurs for house connections are connected on the pipe. A hole is cut at the proper point on the side of the pipe and a mold is placed in the interior. Cement mortar is then spread over the mold and the connection piece is bedded in place and a heavy band of mortar is wiped around the joint on the outside. After the mortar is removed the inside joint is finished with a trowel. This method has been found to be entirely satisfactory.

The main advantages of cement or concrete pipe, over vitrified pipe, are:

1. They can be constructed so as to give them an oval or egg-shape.
2. They can be made with practically no variation in size.
3. They can be constructed with a flat, broad and level base.
4. The joints can be made so that the pipe are self-centering and so that the joints will fit so closely that a continuous smooth bore can be obtained.
5. They can be made in many localities where the cost of vitrified pipe is prohibitive.

Oval or egg-shape pipe are very desirable where the flow of sewerage is variable and where the gradient is very slight. The small invert will permit the flow of small quantities of sewerage with minimum friction on account of the wetted perimeter being less than in the circular pipe.

Pipe of such shape are thoroughly self-scouring whenever there is an increase in the flow of the sewerage and besides will effect a decided saving in flushing water. Pipe of this shape cannot be made of vitrified clay, due to the warping in burning.

Mr. George T. Hammond of Brooklyn, while discussing the paper on Monterry Water Works and Sewerage,* strongly advocated the use of egg-shaped pipe, even as small as 8-in. in

* *Transactions*, Am. Soc. C. E., June, 1911.

diameter. In the Monterry System, 8-in. vitrified pipe was used, but its capacity was much greater than necessary at present. Mr. Hammond says:—

The writer's experience with concrete pipe, derived mainly from a long service in sewer design and construction in Brooklyn, N. Y., leads him to believe that at Monterrey the whole sewer system might, with advantage, have been built with concrete pipe, using an egg-shaped pipe with an area slightly larger than an 8-in. circle designed for a discharge equal to an 8-in. pipe for all the smaller sewers. The invert of such an egg-shaped pipe would fulfil the present requirements in carrying a very small flow with good flotation depth, better than would a 6-in. circular pipe, and the reserved capacity of the 8-in. pipe would be secured without interfering with good present service. Egg-shaped pipes, similar to those used in Brooklyn, the writer believes, would have given far better satisfaction throughout the Monterrey sewerage system than circular fire-clay pipe, and would have cost no more, but probably less. The egg-shaped pipe referred to is made with a flat base and a self-centering joint, thus insuring perfect alignment, and a smoother interior surface than can be obtained with fire-clay pipes.

Cement pipe, in curing, retain accurately the shape given to them by the molds. Vitrified clay pipe, especially in sizes from 15 in. and up, are considerably warped, due to the burning.

In Brooklyn and many parts of Europe concrete sewer pipe are molded with a flat level base. Such a base permits the pipe to rest on a flat foundation, either earth or plank, and thus permits of better back filling. Improper back filling is the cause of much breakage in circular pipe, not supplied with a flat base.

The joints of cement pipe should be so designed that the hub and socket will accurately center themselves. This can be done in cement pipe by making the socket with slightly flaring ends, thus causing the hub, when driven home, to come to its proper position. The vitrified pipe, on account of the warping in burning, must have wide sockets. Such wide sockets are the cause of the off-setting at each joint wherever they are laid. These jogs, or off-sets, collect solid matter in a sewer, and, of course, retard the flow very much. When small openings are used, such as can be made with cement pipe, this difficulty is entirely eliminated and a close and smooth joint is the result. In many places no cement at all is required in the joint as the silt in the sewage quickly fills up the small opening.

In many sections of the country vitrified pipe can only be shipped at great expense, and, wherever, under such conditions,

concrete materials can be obtained, it is far cheaper to make sewer pipe of cement. In many places of Europe, where there are no clay deposits, concrete sewer pipe has been used for years without any question whatever as to its suitability for sewer purposes.

DURABILITY.

In 1877, when Julius W. Adams, Past-President Am. Soc. C. E., was Chief Engineer of the City of Brooklyn, there was a proposal made that the use of earthenware pipe should be resumed. This proposal was adversely reported upon by Mr. Adams. The following are some quotations from his report:

> Last winter there were careful inspections made of our pipe sewers. In one sub-division the grades of streets are over 2½ ft. to the one hundred, and a special examination was made of the pipe in this sub-division. There is no sign of disintegration or wearing away of the cement pipe. These pipe have been laid over five years. On the 25th of March, 1873, this department took up a cement pipe that had been laid in Fleet Street in the year 1861, and the pipe was found, in every particular, as good as when laid—The fact remains, however, that the renewals of sewer pipe in Brooklyn, on account of breakage or collapse, have been relatively less in the case of concrete than in that of earthenware pipe.

Since the above was written, many more miles of cement pipe have been laid, and, as stated earlier in this paper, over 400 miles of cement pipe are in use in Brooklyn. The experience of Mr. Adams has been confirmed since that time. The cost of maintaining concrete sewer pipe has been proven to be less than that of vitrified pipe. This includes renewals, due to breakage, action of acids and so on. It might be of interest to note, that, a few years ago, during the construction of the Subway in Fulton Street, Brooklyn, more of the Fleet Street sewer pipe, referred to by Mr. Adams, was taken up and was found that it would be still good for a long time. The sewer was then about 50 years old. A few lengths of this pipe are in our possession. It was manufactured of natural cement, Cow Bay sand and gravel.

The main argument against the use of concrete pipe is that it cannot withstand the effect of sewerage, nor resist the action of acids. In Brooklyn but two cases have been called to our attention where there has been a failure of cement pipe, due to concentrated acids being admitted in the sewers. Inasmuch as

such sewers have been in eixstence for so many years, the percentage destroyed is entirely neglegible. The author desires, at this point, to make the statement as an Engineer, that if it is desirable to install a sewer which is to take concentrated acid discharges from manufacturing concerns, he would not advocate the use of cement pipe; but would suggest, for such conditions, vitrified pipe of the best quality, with joints of some asphaltic or acid resisting nature and that manholes be built of some other material than brick, laid in cement, because, strong acids affect not only cement, but bricks. It seems to be folly to dwell upon such a contingency. No manufacturer is going to be so extravagant as to permit highly concentrated acids to be wasted. They are too valuable.

Mr. Adams, in his report of 1877, says:

> There has been considerable written about the acids acting injuriously upon cement pipe, but these acids in sewerage matter are greatly diluted, and I am convinced by the experience in this city that they will seldom destroy a good cement pipe. We have known of no case in this city where the street sewer has been disintegrated or eaten away by sewer acids or gases.

Mr. A. J. Provost, in *Municipal Journal and Engineer*, Vol. 20, Page 388, says:

> The argument advanced against concrete sewers is still confined to possible disintegration by acids. The fact remains, however, that the renewals of sewer pipe in Brooklyn on account of breakage or collapse have been relatively less in the case of concrete than in that of earthenware pipe.

Mr. Rudolph Hering, Consulting Engineer, in a report dated February 15, 1908 to the Honorable Bird S. Coler, President of the Borough of Brooklyn, says:

> Portland cement used for the manufacture of concrete pipes is attacked by certain strong acids, such as sulphuric acid, which converts the carbonate into sulphate of lime, otherwise called gypsum, which is comparatively soft and easily eroded. Therefore, cement pipe cannot be used where strong acids are known to enter the sewers.
>
> Vitrified pipe will stand the effect of most acids. When the joints are made of cement these will, of course, be acted upon by acids as readily as the cement pipes.
>
> It should be added that the acid question should be viewed in a reasonable light. When the dilution of sewage is sufficient the discharge of a small amount of even strong acid will not cause objectionable effects, as evidenced by European cities where the use of concrete sewers is almost exclusive in some cities, as Paris and Vienna. In England concrete sewers are also very common. Strong acids attack not only cement, but bricks. Yet bricks are

extensively used both in America and in Europe for building large sewers where naturally any acid would be very dilute.

The greasy substance which is usually found to coat the perimeter of a sewer under the water-line tends to protect the cement from the action of acids to some extent.

It should be added that just as it is objectionable to discharge exhaust steam into sewers and the waste from gas works, which many cities prohibit, so it should be prohibited to discharge strong acid waste into the same.

The Joint Committee on Concrete and Reinforced Concrete of the American Society of Civil Engineers, American Society for Testing Materials, American Railroad Engineering Association, and the Association of American Portland Cement Manufacturers, in its report presented at the Annual Meeting of the American Society of Civil Engineers, January 29, 1909, says:

> Concrete of first-class quality, thoroughly hardened, is affected appreciably only by strong acids which seriously injure other materials. A substance like manure, because of the acid in its composition, is injurious to green concrete, but after the concrete has thoroughly hardened, it satisfactorily resists such action.

Much more could be written about the advantages and durability of cement pipe; but it would be largely reiterating what has been already written about it by others and enumerating the many places both in Europe and America where cement pipe has been in use for years.

The author has positive knowledge that over 400 miles of such pipe laid during a period of over 50 years are giving eminent satisfaction in Brooklyn and in his opinion all arguments against its suitability for sewers are answered by that fact.

THE MANUFACTURE AND USE OF CEMENT DRAIN TILE.

By Charles E. Sims.*

The manufacture of cement drain tile as a commercial proposition began in 1905 when the first power driven machine was set up in a factory in Graettinger, Iowa. Tile had been made for many years previous to this by tamping damp concrete or by pouring a very wet mixture into molds, and tile have been found which were put into the ground at Farmer City, Ill., in 1872; at Ames, Iowa, in 1879; at Monterey, Minn., in 1883; at South Bend, Ind., in 1892; and in many other places of later date. Cement sewer tile have been found used even longer in many cities of both Europe and America. Since 1900 manufacturers of steel molds for making tile by hand have sold an unnumbered quantity of them and tile made in them have gone into nearly every community and into every possible use. In drainage work many thousand tile so made have been laid, particularly of the sizes from 14 to 24 in. diameter—sizes which could be made by this crude method in competition with clay tile.

The first cement tile made by the hand method could not well be of good quality for little was known of the materials and little attention was thought necessary to the manner of curing. It may not be far wrong to say that very few of the tile made in this manner were good quality. Natural cement was often used; dirty or poorly graded sand, or both conditions existing together, was common practice; dry mixtures were insufficiently tamped and no attention was given many times to keeping the tile out of the wind and sun so as to prevent their drying out too soon. Indeed, it is a wonder that the present industry is not carrying on its back a load of failures of such staggering proportions as to prevent its future growth. Tile that were scarcely strong enough to stand the handling, as the result of faulty manufacture, were put into ditches. Strange proportions of sand and cement

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were mixed through ignorance in some cases and through an attempt to economize in others. Strange it is that these tile have as a rule served their purpose.

The abuse of concrete is not confined to the use of hand-made tile, however, as well may be understood when it is considered that the first few years of machine manufacture were years of experiment in machinery, mixtures and methods. The machinery furnished in 1905 and for the next few years was not perfection in any sense, though this is no discredit to the machine manufacturers, for the type of machine had to be developed step by step to meet the requirements of the material. Concrete is not a plastic material and it is therefore impossible to force it into the shape of tile through dies as is done with clay, hence the machinery could not be expected to be best suited to the material in the beginning. The mixtures ranged all the way from 1 part cement and 1 part sand to 1 part cement and 7 or 8 parts sand. Little attention was given to the quality of sand, it being used as it might happen to come from the pit. The mixing was often done by hand or with the continuous type of mixers and a uniform proportioning of cement and sand was not obtained. The amount of water used varied greatly and was usually insufficient. The tile were cured by setting them on racks in a more or less tight building for a day or two where they were frequently dried out before being placed in the yard. A little later steam was advocated thus supplying both heat and moisture, but many experiments were made before it was established that wet steam was essential. These defects in manufacture are not mentioned merely to show the difficulties which were to be overcome, but to show the serious handicap which was thrown upon the material, not to show the general lack of quality but to show the public appreciation of concrete as a suitable material for drain tile and to show the need for a standard of quality.

The growth of the industry since 1905 has been little short of marvelous. Accurate figures are not obtainable but an estimate of the annual production may be worth while, Table I.

At present there are 418 plants which have produced 110,000,000 ft. of tile. Table II shows the distribution of these plants

and the value of their product at the factory; the figures given are but estimates.

Most of the larger sizes have been made in the State of Iowa, most of the first factories were established in this state and more than half the value of all the cement tile manufactured have been produced in this state. Most of the drainage in Iowa is confined within a V-shaped area with Des Moines at the point and the base on the northern boundry line of the state. In 19

TABLE I.—VALUE OF POWER MADE CEMENT TILE FROM 1905 TO 1912.

1905	$20,000.00
1906	80,000.00
1907	250,000.00
1908	600,000.00
1909	1,450,000.00
1910	2,000,000.00
1911	2,600,000.00
Total	$7,000,000.00

TABLE II.—DISTRIBUTION OF CONCRETE TILE PLANTS BY STATES.

State.	Number of Factories.	Linear feet of Tile	Value at Factory.
Iowa	108	42,900,000	$3,750,000.00
Illinois	61	16,000,000	800,000.00
Indiana	60	15,000,000	500,000.00
Ohio	56	12,000,000	480,000.00
Minnesota	30	10,000,000	700,000.00
Michigan	16	3,900,000	175,000.00
Missouri	11	3,000,000	120,000.00
Nebraska	5	1,200,000	55,000.00
Arkansas	7		
Colorado	8		
South Dakota	4	6,000,000	420,000.00
Mississippi	5		
Canada	7		
Other States	30		
Totals	418	110,000,000	$7,000,000.00

counties within this area cement tile have been produced up to 1912 as shown in Table III.

The value of these tile at the plant is estimated at $3,000,-000. In the remainder of the state 70 plants have been built with an output up to the present time of 12,000,000 ft. valued at $750,000. In the year 1911 the product of the Iowa plants had an estimated value of $1,057,000.

Modern methods in the well-equipped factory require first

of all clean, sharp well-graded sand; standard Portland cement; uniform proportions with sufficient cement to fill the voids in the sand—getting away from a weak, porous product as far as possible; water enough in the mix so that the tile will trowel smoothly on the inside and show water marks when the jackets are removed on the outside; that the material shall be packed uniformly hard from end to end of the tile and uniformly throughout the days run; that the packing shall be hard enough so that the newly made tile will resist firm finger-nail pressure without imprint; and that the curing shall be in steam or otherwise dampened atmosphere so that there will be no evaporation from the tile.

TABLE III.—DISTRIBUTION OF PLANTS IN IOWA.

County.	Number of Factories.	Linear feet of Tile.
Cerro Gordo	2	1,200,000
Winnebago	2	350,000
Hancock	2	2,250,000
Wright	4	3,800,000
Hamilton	1	500,000
Boone	3	2,000,000
Webster	2	1,000,000
Humboldt	2	1,250,000
Kossuth	1	2,500,000
Emmett	3	3,000,000
Palo Alto	2	2,500,000
Calhoun	4	1,500,000
Green	3	750,000
Carroll	2	500,000
Sac	1	1,000,000
Buena Vista	2	1,200,000
Clay	2	1,500,000
O'Brien	3	2,000,000
Osceola	1	2,100,000
Totals	42	$30,900,000

Such methods of manufacture are not the haphazard ones of the years gone by and a reliable product must be the result.

The tile manufacturer of today has his own methods worked out or can visit the older plants and find what methods are best but there is no standard of strength or absorption. Until such standards are formulated, manufacturers, engineers and users are in the dark as to what constitutes a good tile. These standards should put all kinds of tile on an equal footing so far as strength requirements are concerned and every factory should possess a testing machine.

The future of the industry will be determined largely by the standard set before it. There will always be those who will never

learn, those who do not care and those who are handicapped by inadequate equipment or capital. There will always be those who advocate that one dollar invested will do where three or five is necessary; that the farmer can make what tile he needs all by himself in his spare winter moments; that a brace and bit type hand-operated machine will produce tile of good quality at a competitive price. Concrete will continue to be abused in some instances but the industry should not suffer in the sight of discriminating men and an established standard will do much to stop this abuse.

The cost of manufacture of cement tile of the 10 in. and larger sizes is less than of clay tile but the transportation charges are higher because the weights are greater. Cement tile plants have the advantage of a smaller investment and can therefore depend upon a smaller territory than the clay plant, hence shipments are confined to short distances. It may, therefore, be expected in future to find cement tile factories in every community where drainage is being actively carried on, supplying the farmer with the best possible tile at the lowest possible price.

DISCUSSION.

Mr. Meade.

MR. JOHN M. MEADE.—It is stated that cement pipe was made in 1861. I would like to ask when the use of reinforcement commenced.

The President.

THE PRESIDENT.—The first important use of reinforced concrete by Ransome in building construction was in 1890. He first used metal in concrete in 1874.

Mr. Meade.

MR. MEADE.—In regard to the disastrous effect of acids, sulphuric acid, on concrete pipes, I know of a case in the last year where sulphuric acid was in the water to be used in the concrete and the chemist condemned it. A man, who did not claim to be an expert, said he would fix it, but the chemist replied that he could not do it, because the sulphuric acid can not be precipitated. This man did not understand the statement. As this was a flowing well, he let the water run into a pond and the sulphuric acid did not precipitate but it did evaporate, and the old bridge man was the best chemist of the two. Now the one thing you have to contend with in this pipe is the sulphuric acid; and it is just in line with this that this occurrence is mentioned to show how such conditions are often overcome.

Reference was made to some large sized concrete pipes, up to 42 in., but it was not stated whether they were reinforced or not.

Mr. Slater.

MR. W. A. SLATER.—Circular bands were placed in some of the forms, not for the purpose of reinforcement but for the purpose of connecting the pipes together, and longitudinal rods were connected with them; it was not considered that they acted as reinforcement.

Some of the pipes are used for drainage and some for sewerage purposes.

Mr. Meade.

MR. MEADE.—An experience which we have had with reinforcement led to some instructive deductions. For a great many years we were making large pedestal stone under the bridges of plain concrete, but when it became the practice to reinforce everything we tried it on these pedestal blocks and a great many

of them have cracked. With plain concrete there never was any trouble. I have been making some investigations and it appears that the reinforcement was placed too near the surface, creating, so to speak, a shell of concrete between the reinforcement and the outside. Experience along this line would be valuable because there has been some question about just where to place reinforcement, *i. e.* how near the surface.

It was stated the time would come when porous tile would be dispensed with. We have discontinued the use of porous tile because the generally accepted theory is that the drainage goes through the joints, and a great many people that use tile really do not understand them, thinking the tile must be porous or it will not absorb water.

MR. CHARLES E. SIMS.—At the present time there is being used a large quantity of the larger size cement drain tile. As an instance I might state that in my own community there is one ditch which calls for 42-in. diameter tile, 3700 ft., to be laid at an average depth of $17\frac{2}{3}$ ft. and a maximum depth of $21\frac{1}{2}$ ft. There is another ditch calling for 32-in. tile. There are no standards at the present time for these different sizes, and I would like to show what the result is in the minds of the engineers who have control of these ditches. One of the engineers in northern Iowa is specifying that the larger size tile shall have a thickness equal to $\frac{1}{10}$ of the diameter and that where they go in the ground over 10 ft. they shall be reinforced against crushing with No. 7 wire, two wires in what might be called one reinforcement, an inner wire to be placed $\frac{1}{2}$ in. from the inside of the tile, the outer wire to be placed $\frac{1}{2}$ in. from the outside, and the two wires to be tied together with light wire; this system of reinforcement to be introduced every 8 in. in the length of the tile. Another engineer specifies that all tile from 22 in. up shall have a thickness equal to $\frac{1}{10}$ of the diameter, with no reinforcement. Another engineer in the same locality specifies that all tile shall be of a thickness of $\frac{1}{12}$ the diameter up to 28-in. sizes and $\frac{1}{10}$ for larger sizes. Engineers and manufacturers feel the need of proper standards. Mr. Sims.

On the ditches where the engineer is requiring the wire reinforcement—and I may say that the same tile is being called for in other sections of the country—the manufacture of the tile is difficult, as the introducing of those wires requires us to stop our

Mr. Sims. machinery to put the wires in and start up again. The result is that the production is materially reduced. Engineers should therefore specify such reinforcing only when convinced of its necessity.

If we have to reinforce the tile it means that a good many jobs are going to go in with vitrified pipe. Many engineers are introducing strict specifications as to concrete tile because they are—shall I say afraid of them?—but let the matter of vitrified pipe alone, accepting whatever the vitrified pipe manufacturer furnishes. That is not entirely fair to the cement pipe manufacturer and yet what could you do if you were an engineer without standards to refer to?

The tile manufacturers today are ready for a standard. I will say that today the tile manufacturers know what good concrete should be. They have the machinery to produce it, if they will only get right down to business and watch what they are doing. We must have dense concrete. I would like to emphasize that; it will be the burden of my talk this morning. If we have a $\frac{1}{16}$ wall, properly made, we can make it so dense that there is no chance for any water to get through, we have a tile that is superior to anything of any other material that is made, and we have a tile sufficiently strong for any purpose today. I do not believe after we get over 10 ft. of earth fill on top of the tile that there is any material increase in the weight upon the tile for any extra depth; and I believe $\frac{1}{12}$ wall thickness for all sizes from 2 ft. up to 30 in. diameter is ample for machine made tile.

In the present proposed specifications there are a few points I do not believe are definite enough. For instance, there is no limit placed upon the size of aggregates. I notice it says that the coarse aggregate shall be considered to be that which is retained on a $\frac{1}{4}$-in. screen, but there is no limit to the size. Our idea is that we should use no larger aggregate than one-half the wall thickness of the tile.

The sentence covering the matter of proportions is rather obscure. It looks to me as though the amount of cement to be used is the amount necessary to fill the voids in the material. This specification hardly says it that way.

Then in the manner of curing, protection from sun and strong currents of air for a period of at least 7 days is not strong

enough. It seems to me that should include the idea of a tight building, where there is no possibility of air currents from the outside. I do not see any other reasons why the manufacturers of cement tile should have any objections to these specifications. I am very sure that a standard of some sort will help very materially. Mr. Sims.

THE PRESIDENT.—I think there have been made a great many tests on cement drain tile. It would seem that the results should be collated so as to obtain a comparison of the varying strength. The President.

MR. SIMS.—There have been a great many tests made at Ames, at the University of Illinois and by the tile manufacturers. The results could be collated, but I do not think that they would present the true facts of today as I do not believe the tests were made on the same quality of goods as manufactured today. I notice in some of the tests on vitrified and unglazed and on cement tile that the cement tile run very close in strength to the vitrified pipe, and I see no reason why they should not exceed it. Mr. Sims.

THE PRESIDENT.—The great trouble is the difficulty of getting cooperation among the cement tile men. One of the recent issues of a paper devoted to cement contained the report of a meeting, I think in Chicago, where the statement was made by one of the cement tile manufacturers, that the National Association of Cement Users was not the proper place to prepare specifications for drain tile; that the tile manufacturer was the one that should prepare them. Somebody should enlighten the tile manufacturers. Certainly it is a basic condition that no producer can properly specify for the consumer, and the National Association of Cement Users is the best medium for preparing a standard for the consumer that you can get. The tile manufacturers, instead of trying to pull away, should cooperate with the National Association of Cement Users in securing such a standard; I believe if we had some cooperation this year from the tile manufacturers we would get a specification. The President.

MR. SIMS.—I do not agree with the statement that the tile manufacturer should formulate his own standard, but he should work out his own rules for obtaining standard strength. Mr. Sims.

THE PRESIDENT.—There has been perhaps some justification for the criticism that our committee tried to tell the manufac- The President.

The President. turer how to make his tile. The proper course for the consumer is to state his requirements and let the manufacturer work out his own method of meeting them. While I think that objection is well taken, I hope that Mr. Sims may be instrumental in getting the manufacturers to appreciate what we need and the difficulties under which we are working. The committee needs the cooperation of the tile manufacturers, to arrive at a standard. You stated, that what was needed was a standard, and I believe the sooner we get it the sooner we can help the tile industry.

We would like you to submit to the committee some statement of what you think the maximum size should be for various sizes of pipe. Do you have a variable standard or just a standard for all tile for the different size of the aggregate.

Mr. Sims. MR. SIMS.—The idea has been to fix the limit at ½ of the thickness of the wall of the tile. I really believe in most of our tile we will not use aggregates that large.

The President. THE PRESIDENT.—Mr. Sims, to what extent is steam curing now being used in tile plants?

Mr. Sims. MR. SIMS.—I believe there are about forty plants that have an investment of $15,000 or more and all these are steam curing, or so-called steam curing. There are about one hundred plants costing $10,000 to $15,000, where they are steam curing but not manufacturing the larger size tile. The idea of steam curing in most of these plants has been developed to a less extent than called for in the specifications. The manufacturer thinks he is steam curing when he is obtaining anywhere from 60 to 80 degs. and keeping the tile in the house 48 hours, really applying the steam only 12 hours out of the 24. That is not efficient steam curing. I think I would be correct in saying that there are but two score of plants that are actually steam curing on lines which you mean. I do not believe that pressure steam curing will ever be practicable, but that 100 to 120 deg. in our curing rooms, curing rooms of low ceiling and single or double track, is efficient, up to date and will be used more and more.

The larger amount of tile are less than 20 in. There are very rapidly coming into use larger sizes, especially in northern Iowa, southern Minnesota, Illinois and Indiana; the development of the machines for making these larger sizes has done wonders to increase their use. In 19 counties of Iowa there have

been used so far about 800,000 ft. of tile larger than 12 in. I think I am right in saying that in northern Iowa and southern Minnesota there will be used this year 400,000 ft. of the larger tile; and the time is no doubt coming when the whole central Mississippi Valley is going to use large sized tile instead of open ditches. The manufacturing cost of the very large sizes of cement pipe—by that I mean sizes larger than 24 in.—is going to be so much less than heretofore, that they are going to use them in preference to the open ditch. Whenever engineers can get cement pipe laid that will take the place of the open ditch at not more than double the cost of the open ditch, they will use the pipe; and these larger sizes are coming into use rapidly. Mr. Sims.

THE PRESIDENT.—At about what diameter does the clay tile cease to become a competitor of the cement tile? The President.

MR. SIMS.—Where we are called for larger sizes than 18 in. we figure that we are getting the better of the clay tile manufacturer. I had occasion to figure some time ago on 30-in., and found our manufacturing cost was approximately half of what the cost of earthenware pipe was laid down at the nearest station. So we have every advantage on this larger size pipe. Mr. Sims.

THE PRESIDENT.—It is an interesting fact that the market conditions as stated by Mr. Sims are practically duplicated in Europe. In England and Germany the competition between glazed tile and the cement pipe is keen for sizes under 24 to 30 in.; at 18 in. there is perhaps a slight advantage in favor of clay pipe, but upwards of those sizes the glazed tile loses out on price; and cement pipe has an advantage over clay pipe that has to be hauled a long distance. The cement products plants are located in the territory where it is used, so that the freight is very low. As to the cost of manufacture the governing size seems to be about 20 in. and above that it is all in favor of cement pipe. The President.

It seems to me that the committee should make an effort by tests made through the cooperation of the colleges or other laboratories, to determine whether the thickness should be $\frac{1}{10}$ or $\frac{1}{12}$, and certainly not put any greater hardship on the cement tile manufacturer than is necessary.

MR. SIMS.—In defense of the $\frac{1}{12}$ wall thickness I wish to say that in Boone County, Iowa, there have been made 200,000 ft. of sizes larger than 12 in. and there never has been a failure that Mr. Sims.

Mr. Sims. I know of. And I would say this: that if we are going to adopt $\frac{1}{10}$ wall thickness now it is going to be impossible to get back to $\frac{1}{12}$. I do not believe there has ever been any case shown that a $\frac{1}{12}$ wall thickness of any size up to 30 in. has ever given any trouble, unless there has been faulty manufacture, or other avoidable causes.

I do not believe the majority of manufacturers of cement pipe, who are using machinery for manufacturing it and clean sand, are having any trouble and would want to see $\frac{1}{10}$ wall thickness under 30 in. We all believe in furnishing enough material if necessary, but it is a serious handicap to furnish this extra $\frac{1}{10}$ wall, because we not only have the cost of the material and the cost of transportation, but we have the prejudice of the ditchers against such heavy tile to fight against. I do not believe it is right to adopt $\frac{1}{10}$ wall thickness without further investigation.

The President. THE PRESIDENT.—Mr. Sims, is the drain tile to which you have referred used simply for the drainage of land; are these pipes very deeply buried in the ground, and is the surface covering very heavy?

Mr. Sims. MR. SIMS.—The instances that I know of are confined entirely, in this discussion, to drain tile, yes; and the depth that I am familiar with has varied all the way from a few feet down to 20 ft.

The President. THE PRESIDENT.—We have a tendency sometimes to be too conservative. Taking building blocks as an illustration, there was recently brought to my attention building regulations that were proposed for adoption in one of the cities in which the specification was taken bodily from those of this Association, which required a compressive strength in 28 days of 2,000 lb. per sq. in. Then further on in this building law it stated that structures and houses of building blocks should not exceed four stories in height. If you figure up the weight of a wall that high in lb. per sq. in., you will readily see that 2,000 lb. in 28 days compressive strength per block is absolutely ridiculous.

There is also a tendency in this direction with tile. It seems to me a tile should be strong enough to carry the load that is put on it. If it is a drain tile such as Mr. Sims has been describing it should be sufficiently strong to carry the load of the fill over it

The President. when placed in position and to stand reasonable handling. Now to fix a strength that would be required, perhaps, for a pipe to be used under different service would be ridiculous. What the Association wants are first-class cement products, and it does not want to advocate a practice giving a factor of safety of 10 or 12 when it would be more economical and equally serviceable with a factor of 4 or 5. The Association wants to help the makers of cement tile, and therefore we ought not impose unnecessarily severe conditions.

MODERN METHODS OF MANUFACTURING CONCRETE PRODUCTS.

ROBERT F. HAVLIK.*

The concrete products industry is but in its infancy, and in the not far distant future factory-made concrete products will be used more extensively than any other one building material. As proof of this, one need but consider the great strides made in the improved quality and increased use of concrete building blocks, trim, brick, drain and sewer tile and concrete roofing.

QUALIFICATIONS OF FACTORY-MADE CONCRETE PRODUCTS FOR BUILDING PURPOSES.

All concrete products increase in strength with age; something which cannot be said of any other building material. They resist fire and with proper care can be made waterproof. Concrete blocks and ornamental trim can be given any surface treatment desired and at a reasonable cost. The hollow space in concrete blocks keeps the structure at a more uniform temperature, thus making it warmer in winter and cooler in summer than the ordinary brick or frame building.

A very popular use to which concrete is put is what is commonly called stucco. It is usually applied to frame buildings on either wood or metal lath. A good result is obtained when the main structure is of hollow concrete building blocks made with a rough surface especially intended for the application of stucco. Stucco frame buildings are not fireproof in any sense of the word. When stucco is applied to untreated wood lath, it cracks in a short time on account of the swelling or shrinking of the lath. On the other hand, when applied to metal lath to cover one side of the lath only, the inside of the lath is left bare. If the stucco is not water proof, rain will come through and rust out the metal lath, causing the stucco to peel off. Thus a new field has been opened to the concrete building block and block manufacturers are rapidly

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taking advantage of it. While such a structure is higher in first cost than a frame stucco building, yet it is permanent and in the end is cheaper than the latter.

Concrete drain tile and likewise concrete sewer pipe are being used in preference to other materials. Everywhere one can see large sewers erected of reinforced concrete pipe. This is convincing evidence that concrete is well qualified for this work. Concrete roofing tile is being used in various shapes and forms. It is pecu-

FIG. 1.—RESIDENCE AT BARBERTON, OHIO. WALLS AND TRIM OF CONCRETE BLOCKS, WITH BRICK VENEER PANELS.

liarly adapted for this purpose, because it is fireproof, waterproof, and becomes stronger on exposure to the elements.

FUTURE OF FACTORY-MADE CONCRETE PRODUCTS.

All over the country concrete trim, often called Art Stone, is being used extensively in place of cut stone and terra cotta. Fig. 1 shows a residence, the walls and trim of which are of concrete blocks, the brick panels merely being used as a veneer, for effect

only. Fig. 2 shows a concrete block railway station. The fact that these products are made in small units, permitting of close inspection, so that inferior or defective products may be rejected, will redound to their advantage.

Several years ago nearly all good concrete trim was made by the cast process, but in the last few years this has been made by the molded process. The principal reason for this is the difference in cost, and as this is always a prime consideration when the quality

FIG. 2.—RAILWAY STATION AT ELKHART, IND. BUILT ENTIRELY OF CONCRETE BLOCKS.

is the same, the molded process is destined to displace the other almost entirely. However, regardless of how the product is made, all that has been said in the above applies to all concrete products, no matter how made.

PROCESSES OF MANUFACTURE.

There are two distinct processes of manufacturing concrete products, one the wet or cast process and the other the molded

or semi-wet process. In the first, the concrete is made wet enough so that it can be poured readily but not so thin as to cause segregation of the materials. The concrete is poured into sand, gelatin or iron molds. In the second process, the concrete should be made so wet that when troweled a little, it will show slight moisture under the trowel. In this case, the concrete is pressed or tamped into iron, wood or plaster molds and the molds are usually removed immediately after the product is made. The first process tends to cause hair cracks and checks in the work. When the product is cast in sand molds, it is expensive, as the mold can be used to produce only one piece, whereas in the semi-wet process the mold is used over and over again. Regardless of what can be said of either process, the semi-wet process is bound to be used in preference to the other, not because the cast concrete is not as good as the molded concrete, but because the latter is as good as the cast concrete and can be used at a much lower cost. Where one piece of work is produced by the cast process, there are probably ten to twenty jobs executed by the molded process. Regardless of the causes, the modern tendency is to use molded concrete products in preference to the cast products.

MACHINERY.

There are two types of machines on the market for producing concrete products by the molded process. In the one, the concrete is pressed into the mold box, whereas in the other, the concrete is tamped into the mold box. Much has been said about the amount of water that can be used in the concrete with the two types of machines. The facts of the case are that as much water should be used in either machine as will allow the product to retain its shape after it has been removed from the machine. It is apparent, therefore, that the amount of water will be the same in both cases, whether the product is made in a press or tamped into a mold box. Furthermore, nearly all concrete products are faced with a rich mixture and in order to keep this from sticking to the face plate of the machine the facing must be made drier than the backing.

The principal thing to determine in considering what machines to use, is the output of the machine per day per man, or the cost of producing the blocks on the two types of machines, all other

things being equal. Cost of production, wear and tear, and the power required, determine the final cost of the products, and yet all these points are seldom given the attention they deserve by the prospective buyer.

The tendency today is to manufacture blocks with automatic machinery as far as possible—and this has forced machinery manufacturers to produce automatic tampers for tamping blocks, automatic chargers and elevators for feeding the materials into the machines, and various other automatic devices for increasing the output of the machine and decreasing the labor.

STRENGTH OF CONCRETE PRODUCTS.

As concrete products are being used more extensively than ever before, the building authorities in larger cities are demanding that they meet certain strength requirements. This is a good movement which will do more than anything else to improve the quality of the product placed on the market and also insure the public as to its safety for building purposes. The usual requirement is that concrete building blocks have a minimum average crushing strength of not less than 1000 lb. per sq. in. of gross cross-sectional area of the product. As the average air space in hollow concrete building blocks is close to 33 per cent, this figures to 1500 lb. per sq. in. of net cross-sectional area of a building block. Solid products, such as sills and caps, should have a minimum crushing strength of 1500 lb. per sq. in. of net area, for the reason that if this were only 1000 lb., the sill or cap might be weaker than the blocks on which it rests, as oftentimes the hollows of blocks under caps are filled in solid with concrete.

In order to get this strength, it is necessary to use good material and a fair amount of cement. Where trouble is experienced in this regard, it may be necessary to increase the amount of cement or else grade the aggregates, whichever of the two costs less.

GRADING OF AGGREGATES.

The most common aggregates are sand and gravel, but crushed stone is often used in place of gravel, and stone screenings in place of the sand. The latter should not be used unless the fine dust is screened out, as in almost every case crushed stone screenings contain as much dust as they do coarse material;

this dust weakens the concrete. In some instances concrete block manufacturers were required to make blocks that would crush at not less than 1000 lb. per sq. in. of gross cross-sectional area. They used soft limestone screenings with a high percentage of dust and mixed nothing with this but a little Portland cement. It is self-evident that blocks could not meet this requirement when made of such materials and yet the manufacturers could not understand why this was so. On advice one discarded the screenings and used sand and gravel instead, and, as a result, increased the strength of his product over 50 per cent.

Sand and gravel makes better concrete than screenings and crushed stone, as the former compact much better. Crushed stone and screenings arch readily and thus cause voids in the concrete.

If the aggregates are fairly well graded as they are purchased in the market, it is sometimes just as economical to use a slightly larger proportion of cement as it is to put in expensive machinery for elevating and grading them. It is especially true where the manufacturer is limited in capital. An additional amount of cement does not materially increase the cost of the product. However, if the manufacturer can afford it, he should put in the proper bins, elevating and screening machinery, as he will then be sure of a uniform product and will not have to test his aggregate constantly to determine the quantity of cement needed.

There are three principal methods in vogue for determining the proper proportions of each aggregate that should be used. The first is the "void" method, the second proportioning by trial mixtures, and the third, proportioning by the study of the mechanical analyses of the various aggregates. The first is universally acknowledged to be a little better than a guess. The second is a very good method to determine the best proportions to use of the various aggregates just as they are. The third method, however, quickly shows the same results and also the quantity of any size aggregate required to produce the best concrete.

In the third method, the aggregates are screened through various size screens, and the percentages by weight of the materials passing screens of certain meshes are carefully recorded. For any given amount of cement, the strongest concrete is produced from that combination of aggregates whose mechanical analysis

TABLE I.—THIS TABLE SHOWS THE PERCENTAGES BY WEIGHT OF ANY SIZE AGGREGATE REQUIRED FOR AN IDEAL MIXTURE, THE LARGEST AGGREGATE OF WHICH CORRESPONDS WITH ONE OF THE SIZES GIVEN.

For example: if the largest stone is 1 in. diameter the percentage of each of the other sizes is found in column 3.

The material smaller than .015 in. includes the cement. This data is figured by the formula $P=100\sqrt{\frac{d}{D}}$

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
Diameter of stone in inches.	Per cent smaller than diameter to left.	Per cent between diameter to left and next larger diameter.	Per cent smaller than diameter to left.	Per cent between diameter to left and next larger diameter.	Per cent smaller than diameter to left.	Per cent between diameter to left and next larger diameter.	Per cent smaller than diameter to left.	Per cent between diameter to left and next larger diameter.	Per cent smaller than diameter to left.	Per cent between diameter to left and next larger diameter.	Per cent smaller than diameter to left.	Per cent between diameter to left and next larger diameter.	Per cent smaller than diameter to left.	Per cent between diameter to left and next larger diameter.	Per cent smaller than diameter to left.	Per cent between diameter to left and next larger diameter.
1.00	100.0	0.0														
0.75	86.5	13.5	100.0													
0.50	70.6	15.9	81.6	18.4	100.0											
0.30	54.7	15.9	63.3	18.3	77.5	22.5	100.0									
0.25	49.9	4.8	57.7	5.6	70.6	6.9	91.4	8.6	100.0							
0.20	44.7	5.2	51.6	6.1	63.3	7.3	81.6	9.8	89.5	10.5	100.0					
0.15	38.7	6.0	44.7	6.9	54.8	8.5	70.6	11.0	77.4	12.1	86.5	13.5	100.0			
0.10	31.6	7.1	36.5	8.2	44.7	10.1	57.7	12.9	63.2	14.2	70.2	15.9	81.6	18.4	100.0	
0.05	22.3	9.3	25.8	10.7	31.6	13.1	40.8	16.9	44.7	18.5	50.0	20.6	57.8	23.9	70.6	29.4
0.02	14.1	8.2	16.4	9.4	20.0	11.6	25.8	15.0	28.3	16.4	31.6	18.4	36.5	21.2	45.7	24.9
0.015	12.2	1.9	14.2	2.2	17.6	2.4	22.3	3.5	24.5	3.8	27.4	4.2	31.6	4.9	38.7	7.0
Smaller than 0.015		12.2		14.2		17.6		22.3		24.5		27.4		31.6		38.7
Total		100.0		100.0		100.0		100.0		100.0		100.0		100.0		100.0

plotted to a curve by the co-ordinate method forms a parabola, passing through the zero ordinates and the intersection of the diameter representing the largest stone with the 100 per cent ordinate. This method was discovered by Mr. Wm. B. Fuller* in 1901. The percentage by weight of the aggregate smaller than a given size can be easily calculated by the formula for the parabola:

$$d=\frac{P^2D}{10{,}000} \text{ or } P=100\sqrt{\frac{d}{D}}$$

in which P = percentage of mixture smaller than any given diameter; d = any given diameter; D = largest diameter of stone used.

Table I gives the percentages by weight required of any size aggregate to form an ideal mixture.

As a rule, concrete aggregates have too much fine material and if the aggregates are screened and graded as above, there is a surplus of the fine material which may be disposed of in some manner as it keeps accumulating and this feature should be given careful consideration in designing a grading plant.

MIXING.

When the selection of the aggregates is properly taken care of, the next important consideration is that of properly mixing the concrete. The superiority of machine mixing over hand mixing is an accepted fact and needs no argument. Mixers should be used by all means and none but power machines, as hand mixers are man-killers as anyone can testify who has used them.

At present there are two types of mixers on the market: the batch and the continuous, self-proportioning machines. As both types are well known, they need no further description. At the New York Convention† there was considerable discussion of the variation caused in the mixture by the moisture in the sand. It was there recommended that the materials should be proportioned accurately by the use of measuring-boxes, chutes or some such device, the inference being made that proportioning mixers do not do this. The general impression gathered was that

* See Taylor's and Thompson's, *Concrete, Plain and Reinforced.*

† See *Proceedings*, Vol. VII, p. 793.—ED.

if the materials were measured in some such way as this and mixed in a batch mixer, the results would be almost perfect. However, it does not seem that it was the intention of the speaker to create this impression. There are many, however, who believe that this is true. A good continuous mixer will handle wet materials just as well as a batch mixer and yet neither one of the two is ideal. Practically all continuous proportioning mixers measure the materials, the only difference being that a continuous mixer measures the materials in smaller quantities than is done with a batch mixer.

The only danger to contend with in a continuous mixer is that wet materials may arch and not feed properly, but this is not due to a mistake in the principle of the machine, but rather in the design. The sides of the hopper must be steep and the bottom large enough so that the materials cannot arch. These features should be looked into carefully when purchasing a machine of this type.

It was also brought up at the last Convention that moisture in sand affects its volume materially. No one who knows anything at all about concrete will dispute this fact, but the moisture affects a large batch just as much as a small batch and, therefore, a large batch of wet sand will affect the resulting concrete when mixed in a batch mixer just as much as a small amount will affect the concrete mixed in a continuous mixer. It is evident, therefore, that whether the materials are mixed in a continuous or batch machine, they are as likely to be affected by the moisture in the sand as much in one machine as in the other. Neither is perfect, but the only way to get a perfect mix is to dry all the aggregates and then either type of machine will handle them satisfactorily. It has often been suggested that the aggregates be weighed. There is no doubt but that this can be done, but it cannot be done economically with any machines on the market today.

It has been suggested that mechanical proportioners should be prohibited unless they proportion by weight only. This would be fair if the same were required of batch mixers, but to require this of both machines, would find no machines on the market to fill the requirements.

In discussing this question, a tile manufacturer plainly stated that although using a batch mixer for dry mixing and a paddle

conveyor for wet mixing, that the mixtures would vary materially on account of the moisture, in fact, as much so as in a continuous mixer and, oftentimes, instead of getting a 1 : 3 mixture, it would be a 1 : 2 or 1 : 2½. This proves that neither type of machine does this work satisfactorily unless proper precautions are taken to correct the mixture for the moisture. If these precautions are taken, both types of machines will mix very satisfactorily.

PROPORTIONS.

All concrete products should meet certain strength requirements and the proportions must be governed accordingly. It is immaterial whether the proportions are 1 : 3 : 5, 1 : 2 : 3, or anything else, providing the products made from these proportions of cement and aggregate meet the strength requirements.

FACING AND SURFACE TREATMENTS.

Nearly all concrete blocks and other products such as brick and trim are faced with a rich mixture of cement and sand. As such products are sold on their appearance, it is essential that the face be very hard and all corners and edges solid. The usual proportions used in the facing vary from 1½ to 3 parts of aggregate to 1 part of cement. When ordinary sand is used, it should be graded from ⅛ in. down to a No. 50 mesh, with no material smaller than No. 100 mesh. In white facing, the proportions must be at least 1 part of cement to 1½ to 2 parts of aggregate, as all available white aggregates are either all one size, as in the case of white sand, or contain too much dust, as is the case with most crushed white marble.

The manufacturer can readily obtain any surface effect desired. If he wishes to color his blocks or other products, he can do so by using mineral colors. Should he wish to expose the aggregate used in the facing, he can do so by scrubbing the face of the product with acid and water after it has set for 24 or 48 hours, or by spraying it with a fine spray of water just as soon as it is made.

Many architects have desired a polished concrete product. Concrete blocks can be polished so that they will have a face closely resembling terrazzo. The facing mixture used is practically the same as that used in terrazzo floors and when the product

has hardened sufficiently it can be polished with polishing machines or by hand. Hard polishing, however, makes the cost prohibitive and even with polishing machines the cost is more than the average user cares to pay. For that reason, polished concrete products will not be used extensively until cheaper methods of polishing have been discovered.

For many years, concrete building blocks have been condemned on account of the so-called rock face. This style of block has probably hurt the business more than anything else, but yet the tendency today is toward the use of other designs, so the product should no longer be condemned on this account, but its use rather promoted by praising where praise is due, and certainly the vertically and horizontally tooled, bush hammered and plain blocks deserve favorable comment.

CURING.

Concrete products are cured usually by either of two methods. The more common method is to sprinkle the blocks with water as often as necessary to keep them moist constantly for a period of about 10 days, after which they are stored in the yard and allowed to cure naturally. The second method is to expose them to an atmosphere of saturated steam at atmospheric pressure and allow them to cure thus for 48 hours, after which they are stored in the yard until used.

The modern and best method is the second, as blocks so cured can be readily handled the third day after they are made without danger of damaging the edges or corners and they can be used much earlier in a building than if cured by sprinkling.

Extensive tests with high pressure steam show that the strength of a concrete product is materially increased by curing in an atmosphere of high pressure, saturated steam, similar to what is used in curing sand lime products. It is very probable that in the not far distant future nearly all concrete products will be cured in this manner. Blocks so cured are ready for the market in 48 hours, and a material saving in cement can be affected. The first cost of such a plant, however, is quite high and beyond the means of the ordinary concrete products manufacturer.

The modern tendency, however, is to centralize this industry and manufacture concrete products on a large scale, which of

itself will bring this condition to pass. The tests referred to above prove conclusively that there is no danger from too high a temperature, providing the steam is saturated. A simple fact little understood by the user of concrete is that steam in the presence of water is always saturated, regardless of the pressure. As soon as it is drawn from the boiler, it commences to lose heat, the temperature is lowered, and consequently the steam loses moisture in the form of condensation. In a steam curing plant, the steam is taken directly from the boiler and fed into the kiln. The kiln is not heated by external means and therefore is always at a lower temperature than the steam at the boiler, consequently the steam loses heat as soon as it enters the kiln and condensation takes place. It cannot under any condition extract moisture from the blocks or other products unless a fire should be built under the kiln and the kiln heated to raise the temperature above that of the steam in the boiler.

This is one of the first principles of steam engineering, and it is wrong to state that the temperature of the steam curing kiln must not be over 100 deg. or so Fahr. Instead of maintaining the temperature of the curing kilns at this point, it should be raised as much as possible by the steam itself as fed from the boiler and the resulting product will be far stronger. The feeding of the steam through a pipe which is immersed in water is unnecessary, although in the writer's first tests this was done, but more because it was pointed out that unless these precautions were taken the product would go to pieces. The best plants today feed the steam direct into the kiln, without immersing the feed pipes in a trough filled with water. There is no harm in this method, but it is altogether unnecessary.

No concrete products manufacturer should cure his product by sprinkling unless he positively cannot afford to put in a steam curing plant, as the latter is the only up-to-date, modern method.

WATERPROOFING.

In speaking of waterproofing regardless of what can be said of monolithic concrete, concrete building products such as blocks and trim should all be waterproof, so they will absorb little or no water. It has been frequently contended that they can be made waterproof without the use of a waterproofing. Considerable

experience along this line, failed to make a waterproof block without the use of a waterproofing, regardless of how wet the facing. It is contended that concrete building products should be dry in a few hours after a severe rain, and no product made without a waterproofing will do this.

Blocks can be made waterproof to such an extent without waterproofing that moisture will not effect the interior of the building, but that is not sufficient, for such blocks retain moisture in the face entirely too long. It is this very feature that creates the general impression that concrete products are not waterproof and for that reason alone, if for no other, a reliable waterproofing should be used to prevent it.

The great majority of waterproofings now on the market are unreliable, but yet there are a few that are very good. Much of the adverse criticism of waterproofings is due to laboratory tests made on small specimens, perhaps 2 in. cubes. Such tests are unreliable from personal experience. With any waterproofing, a sufficient amount of water must be used to show a little moisture on the surface of the product when made, or else it will not be waterproof. Another precaution that must be taken is to tamp the concrete very thoroughly. It is impossible to do this in small laboratory specimens and that is why they seldom prove favorable to the waterproofing.

In making full-sized blocks, there never was any trouble in producing a waterproof block when observing the above precautions, and yet in making quarter-size model blocks, trouble occurred until the faces were tamped very thoroughly.

ARRANGEMENT OF PLANT.

The arrangement of a concrete products plant should have very careful consideration. The main objective should be to keep the cost of production as low as possible. To do this, it is necessary to handle the raw materials as economically as possible and also the finished product. The plant should be so laid out that screening apparatus can be installed at any time without seriously affecting the general lay-out of the plant. Overhead bins should be used if possible. These can be elevated up to the second floor, so the mixer can be fed direct from the bins and the concrete fed from the second floor to the machines. If the bins are of large

capacity, they will run up in height a great deal. Should this prove an objection, they can be placed so as to discharge on the first floor into an elevator which elevates the material into the mixer on the second floor, and the discharge gates of the bins can be handled from the second floor by the man operating the mixer. The concrete is dumped into a dump car, which discharges into hoppers located above the machines. When it is impossible to use this arrangement, the mixer can be located on the first floor, and discharge into a portable dump bucket, which can be operated on an overhead rail. All of the machines and molds should be located on one side of the plant. The cars should be directly in front of the machines or molds. At Barberton, Ohio, the car tracks are set about 9 in. lower than the surrounding floor, so that the cars stand in pits and the first deck is a few inches above the floor level.

This is a very excellent method, as the block-maker is enabled to load the highest deck without strain. When the cars are loaded, they are run onto a transfer car and then into the kilns. Two pits should be used for each machine, so that as soon as one car is loaded the block-maker can commence loading the second car while the first is being run into the kiln.

When the blocks have been cured for 2 days, they should be stacked in the yard as conveniently as possible for loading on wagons or railroad cars. No two plants can be designed alike unless the available ground space is approximately the same. For that reason, one can only offer suggestions as to the general scheme and each plant should be figured out by itself, making use of these principles as far as possible.

The steam curing kilns should have semi-circular ceilings so that the condensed steam will run down the sides of the kiln and thus not stain the concrete products, as is the case where flat ceilings are used. The best method of feeding the steam into the kiln is probably through a pipe along the floor in the center of the kiln, so that the steam may escape to the side of the kilns, rise upward, and circulate to the roof where the two streams meet and are forced downward again. Proper arrangement should be made for draining the condensed steam back to the boiler or a pool.

BUILDING LAWS.

All the large cities are adopting building laws governing the use of concrete products and this question should be considered very carefully and reasonable regulations drawn up which can be presented to the various building departments. Laws are certainly necessary for this purpose, but they should be uniform and fair to both the producer and user of the product. All hollow concrete building blocks should be required to have a minimum average crushing strength of 1000 lb. per sq. in. of gross cross-sectional area of the block, and solid products should be required to have 1500 lb. per sq. in. figured on the same basis.

This will insure a uniformity of the product and the thicknesses of walls should then be figured accordingly, regardless of what is allowed for other building materials. For blocks laid up in a lime and Portland cement mortar, a load of 167 lb. per sq. in. of gross area of the wall should be allowed, and when they are laid up in a Portland cement mortar a loading of 200 lb. per sq. in. will be safe. In both cases, the factor of safety will be at least 5. On this basis, the thicknesses of walls of average buildings can be readily figured and recommended as a standard. All other buildings not covered by this should be figured on the above basis. When the hollow spaces are filled in solid with concrete and the blocks are laid in a Portland cement mortar, 300 lb. per sq. in. can be safely allowed.

This phase of the concrete products industry is very important, and should have immediate attention by the Association. Concrete block manufacturers everywhere are asking for information on this subject, so they can present data to their local building authorities. Heretofore our regulations have been too general, and referred this matter to what is allowed for other building material. This is radically wrong, for, knowing the strength of the material, it should be figured on the above basis only and to refer it to other building materials implies that the strength is not known.

Regulations of this kind should be adopted by all cities, but they should not be too severe so as to kill the industry. If once adopted, however, it will place the concrete building block at an advantage over any other material, as others are not required

to meet any test requirements and therefore the walls are usually figured very heavy to guard against the poorest product that may be on the market.

COSTS AND PROFITS.

The costs and profits of this business deserve careful attention. The concrete products industry offers a better field for the investment of capital than most industries, for the reason that it is in its infancy, the demand for concrete products is increasing constantly and because it has an unlimited field.

However, as in any other business, the production costs must be watched very carefully. The overhead expense must be watched even more so. A small firm cannot afford to have more than one salaried officer who is not actually producing the product. Too many plants have failed because every man interested therein desired an office and a nice salary, and did little but sit at a desk and draw his salary. The actual productive cost is usually such as to admit of a very nice profit, if there is a volume of business and the overhead expense is kept down.

Under overhead expense, everything should be included, such as salaries of officers, office expense, power, fuel, light, water, taxes, insurance, interest on investments and depreciation, and any other items that cannot properly be classified as actual labor and material cost of the finished product. This is never known in exact dollars and cents on any one product, but must be estimated as a percentage of the actual productive labor cost, and added thereto and to the cost of materials to arrive at the selling cost of the product. Anything over and above this that is obtained in the selling price will be the net profit on the product and will not usually be 5 cts. per block, as many manufacturers claim, but more likely 3 cts. or so. If the manufacturer does not draw a salary, his worth is a certain salary per year, a part of the overhead expense, and the balance of the profit will be a clear, net profit.

REQUISITES OF BUSINESS.

No firm or individual should attempt to go into the concrete products business without a sufficient amount of capital to buy the necessary machinery, a stock of cement, sand and gravel

and a surplus to do business with. The exact amount required depends on local conditions and the extent to which it is intended to manufacture concrete products, but in no case should it be less than $1500 to $2000 if it is desired to stay in the business long enough to get a firm footing. No one can go into this extensively without a capital of $6000 to $10,000, and in that case should merely rent or lease the ground on which the plant is located, or buy it on a long-time mortgage.

The actual amount of capital that is necessary for machinery is not excessive, but the surplus to do business with should exceed the actual money put into machinery and materials. Good advice to either an individual or a firm would be to lay plans for a large plant and then put in only what is absolutely necessary at the start, but arrange the building and machinery in such a way that additions can be made to the plant without affecting the existing plant. In this way, no capital will be wasted. The machinery purchased should supply all immediate needs, but too much should not be purchased at the start. After the business has been started and the manufacturer is familiar with the market, such other machinery as needed can be added to this equipment.

Every firm should have the services of at least one man who thoroughly understands all phases of the concrete products business. This man should have a knowledge of plans and should be able to figure the actual labor and materials cost of each product. Such a man should also be able to handle men so as to get the maximum production.

NOTES ON REINFORCED CONCRETE TELEGRAPH POLES.

By George Gibbs.*

Among the facilities required on the New York Terminal Division of the Pennsylvania Railroad was the railroad company's telephone and telegraph service. The line for this purpose forms an extension of the New York-Philadelphia main line from its point of intersection with the Terminal Division at the east side of the Passaic River, near Newark, N. J., and consists of pole line (Fig. 1) along the right of way across a continuous 5-mile stretch of semi-tidal meadow swamp land, known as the "Hackensack Meadows," to the tunnel portal at Bergen Hill, the Hackensack River being crossed midway of the section. The ground surface is covered with a heavy growth of reeds, and the top stratum is a peaty bog, from 8 to 15 ft. deep, underlaid with varying strata of clay, fine sand, and mixed sand and clay for very considerable depths. It will be seen, therefore, that the problem was to provide a design of pole which would not only be strong in itself, but which could be set in a suitable foundation in a soft and uncertain ground. It was desired to make the line entirely secure against interruption by severe storms or by fires in the swamp reeds. It appeared, under these conditions, that the ordinary type of wooden pole would not be wholly satisfactory, either as regards strength for such a heavy service, or safety as regards the frequent meadow fires.

The use of reinforced concrete poles for the purpose appealed to us, but very little reliable data about either the cost or service from such design was obtainable. In Europe the author had observed the quite extensive use of concrete poles, chiefly for electric transmission purposes, but details of their construction were not at hand. These poles are frequently of structural steel covered with a protective coating of concrete and sometimes are of wood encased in concrete, and much ingenuity has been expended

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in devising special forms of composite poles, but with what practical success it is difficult yet to say.

The desirability, however, of securing a strong fireproof line across this section of our road was so great that the writer undertook to design a simple form of reinforced concrete pole, which could be manufactured on or near the work and at a cost which would not be prohibitive. Without previous data to go upon the design evolved is somewhat experimental and probably the notes here given will not be of very great assistance in fixing a precedent

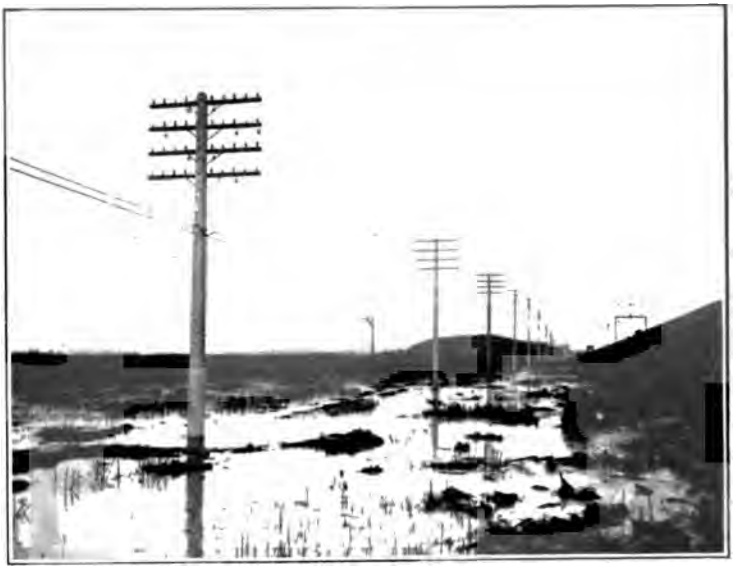

FIG. 1.—CONCRETE TELEGRAPH POLE LINE OVER HACKENSACK MEADOWS, N. J.

for an ordinary telegraph pole, as the conditions call for heavier construction then usually required for such lines. The data, however, is given for what it is worth, as indicating a heavy and difficult piece of construction for a special problem.

DESIGN OF LINE

Two hundred and two poles were required, spaced from 70 to 135 ft. apart, with an average standard spacing of 120 ft.; the variations in span being due to numerous railway and highway

crossings. The height of the top of poles above the ground also varies for the local conditions from 25 to 50 ft. and the total length of poles from 35 to 65 ft. The loading called for an ultimate capacity of 60 open telegraph wires and two 40-pair telephone cables and one signal control cable. The open wires are of copper, Nos. 7 and 8 B. & S. gage, and the cables Nos. 13 and 16 B. & S .gage insulated wires, with a lead covering. The cables are carried by steel messenger wires. The weight of the wires and cables is 10.5 lb. per running foot, giving a total loading for the average span of 1268 lb. per pole, or including cross-arms, insulators, brackets and fittings, 1500 lb. per pole. In addition to this static vertical loading, emergency conditions were assumed of an extra loading of ½ in. of ice on the wires and a wind pressure of 8 lb. per sq. ft. on wires and 13 lb. per sq. ft. on the poles; the stresses under these conditions not to exceed the elastic limits of the wires and of the carrying structures. These assumptions are equivalent to a vertical loading of 7600 lb. per pole and a horizontal force of 4000 lb. exerted one foot below the pole top. The normal sag of the wires at 60 deg. Fahr. temperature is 4.5 in. and a normal tension of 273 lb. for No. 8 wires and 130 lb. for No. 9. The maximum hot sag at 130 deg. Fahr. is 11 in.; the tension in the wires under maximum wind pressure with the ice loading and at −20 deg. Fahr., is 702 lb. and 470 lb. respectively for the two sizes of wires.

The design adopted is that of a tapering smooth-surface, reinforced concrete pole, square in cross section with chamfered corners and having a taper of ½ in. in 5 ft. The concrete is of 1½:2:4 mixture, assumed to have an ultimate strength in compression of 2000 lb. per sq. in. The reinforcement is composed of mechanical bond bars tied together into a square skeleton frame, and in the complete pole this reinforcement is covered by a 1 in. minimum thickness of concrete. Fig. 2 and Table I give details of the complete construction.

In order to manufacture the poles economically a yard was established adjacent to the right of way and with side track connections for delivery of materials. The ground was leveled and set with stringer pieces, 3 ft. apart, of old wooden car sills, so as to furnish a convenient surface upon which to erect the timber forms. This yard permitted of the simultaneous manufacture of 6 poles and the storing in place of 90 poles during the period of

FIG. 2.—MANUFACTURE OF CONCRETE TELEGRAPH POLES, SHOWING SKELETON FRAME SUSPENDED IN PLACE; FRAME IN FORM READY FOR POURING; AND FINISHED POLE.

TABLE I.—DESIGN AND CONSTRUCTION DETAILS OF CONCRETE POLES.

Length in feet.	Equivalent Pull 1 foot from Top, lb.	Edge Distance for Reinforcement, in.	Area Reinforcement, sq. in.	Extreme Fiber Stress in Concrete Compression, lb. per sq. in.	Unit Compression Stress in Reinforcement, lb. per sq. in.	Weight of Pole, lb.	Concrete, cu. ft.	Weight of Reinforcement, lb.	Lineal Feet of Wire wrapped around Pole.
30	4066	1.75	3.38	1953	19,059	5,000	33.15	819	228
35	4321	1.75	3.63	2087	20,480	6,200	41.44	1077	273
40	4505	1.75	4.03	1917	19,667	7,600	50.75	1309	333
45	4647	1.75	4.40	1987	20,428	9,200	61.58	1493	375
50	4763	1.87	4.73	1886	19,692	11,000	73.24	1792	431
55	4860	1.87	5.00	1832	19,155	13,000	86.58	1974	490
60	4945	2.00	5.30	1812	19,083	15,000	101.28	2249	553
65	5020	2.00	5.48	1976	18,591	17,600	117.23	2575	618

seasoning. In the process of manufacture the skeleton reinforcement is first made up, as shown in Fig. 1, the twisted steel mechanical bond bars being tied together to form a square skeleton frame; this frame is then suspended at the proper height above the ground and the sectional wooden frames put in position around it. The concrete mixture is then poured in and carefully tamped. In this way 6 poles were made per day and were left in place at least 16 days to season.

FIG. 3.—SETTING AND JETTING OF CONCRETE TELEGRAPH POLE.

FOUNDATIONS

After a number of experiments it was found best to set the poles in pits excavated in the marshy stratum. These pits were generally about 9 ft. square and 5 ft. deep, and a timber grillage was placed around the base of each pole and about 5 ft. below the top of the ground. This grillage consisted of six track cross-ties bolted together and to the pole, and partly planked over by 3-in. rough lumber. The pole, which projected below the grillage and was pointed at the butt, was jetted down by compressed air into the sandy layer, so that the grillage would rest at the bottom of

the pit (Fig. 3). The pits were then back-filled with rock and clay. Poles on curves are cross-guyed and the terminal and railway crossing poles are head-guyed with steel cables.

In order to determine the stability of the foundations, as above described, as well as the transverse strength of the pole itself, a test was made of a 55-ft. pole set in a 13-ft. deep foundation (Fig. 4). The test consisted in applying a horizontal pull 39.5 ft. from the ground. The breaking load was 4360 lb. and the point of fracture 20 ft. from the ground, at the top of the 32-ft. reinforcement bars. The foundation itself was found to be amply secure.

COST

The actual cost of labor and material for manufacture of these poles, reduced to a unit cost for an average length pole of 40 ft., weighing 7600 lb., was as follows:

Forms—labor and material	$19.00
Concrete—labor and material	26.50
Reinforcement—labor and material	32.00
Total per pole	$77.50

The above does not include cost of preparing site, engineering, etc., and is for the bare poles (without fittings or foundation timbers). To obtain the cost of the poles erected there must be added the cost of loading, distribution, foundations and setting, cross-arms and fittings, but these items were special to the particular location and would be of little use as a matter of general information. It might be stated, however, that the cost of handling and erecting these heavy poles was considerably more than would be the case for wooden poles; furthermore, care must be taken in loading and unloading them and special appliances are required for their erection.

In the special case in question it was figured that the cost of the line erected complete was not greatly in excess of that of an equally stable wooden pole line, as in order to meet such a condition it would have been necessary to employ "H" frame poles and set them in equally expensive foundation pits. In the case of this line it is interesting to note that the foundations and setting cost considerably more than the poles themselves.

To sum up, it may be said that for the particular case of this heavy line, having very difficult foundation conditions, the use of a reinforced concrete pole resulted in a durable, safe and neat appearing construction at little additional cost to that of a wooden pole line inferior in all these respects. For the lighter class of telegraph lines, however, the writer would anticipate that concrete poles would always cost more than the wooden pole construction, as ordinarily designed and erected.

FIG. 4.—REINFORCED CONCRETE TELEGRAPH POLE TESTED TO DESTRUCTION.

Mr. L. V. Morris, Chief Engineer Jamaica Improvement, of the Long Island Railroad Company, has lately designed and manufactured a number of reinforced concrete piles to be used as station platform shelter supports. These piles are 28 ft. in length and probably represent fairly well a telegraph pole of ordinary dimensions and strength. The writer, therefore, gives, with Mr. Savage's permission, some information regarding them as a matter of general interest and for comparison with the heavy poles previously described. These piles are 28 ft. long, of hex-

agonal cross section, tapering from 16 in. at the butt to 8 in. at the top. The reinforcement consists of eight ½-in. square twisted steel rods with a hooping of No. 7 wire wound on spirally, the ½-in. rods being attached to a tapered mandril. At various points the hooping and the rods are wired together to secure the frame and prevent distortion. The piles contain slightly under 1 cu. yd. of concrete and weigh about 3700 lb., or an average weight of 132 lb. per ft. This compares with 190 lb. per ft. for the Terminal Division telegraph poles. One of these piles was recently driven to a depth of about one-half its length and refused to go further, although a thousand blows of a steam hammer were delivered at the head; no appreciable effect resulted from this rough treatment.

CONCRETE FENCE POSTS.

By W. J. Towne.*

A small plant was established at Dunlap, Iowa, in 1907, for the manufacture of concrete posts, the most of which were used in the vicinity of that station.

The plant was of the open-air character, all work being done, whenever possible, in the open air, a small temporary building being erected for the storing of cement. The equipment consisted of 265 molds for 7-ft. regular line posts, and 25 molds for 8-ft. end and brace posts; also one mold shaker with capacity of 12 molds.

Three men were employed, paid from 16½ to 17½ ct. an hour, and they manufactured an average of 60 posts a day. The line posts cost 18.3 ct. each and the end or anchor posts 53 ct. each.

The posts were reinforced with steel strips ⅛ x ⅝ in. x 6 ft. 10 in. long. The mixture of concrete consisted of 1 part cement and 6 parts gravel.

About 5000 posts were manufactured and most of them placed in the territory between Dunlap and Missouri Valley, Iowa, where, owing to frequent heavy floods, the use of wooden posts is almost prohibited. The posts are giving very good satisfaction, but it is recommended that they be made 8 ft. long and have additional reinforcement, in order that they may be less likely to break in handling.

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CONCRETE FENCE POSTS.

By L. J. Hotchkiss.*

A paper on this subject should include a description of the more important types of posts so far developed and a discussion of their merits. It has not been possible, however, in the time available, to get together the proper information, and this paper, therefore, is confined to the one type of post used on the Burlington Railroad.

The post machine was set up in 1911 and a small number of posts, about 6000, were made. In going into a new proposition of this kind there are always some delays in getting the plant organized and operating smoothly. The usual amount of trouble along this line has been experienced and it has not yet been possible to determine how cheaply posts can be made.

Fig. 1 is a line drawing of the plant as now arranged. The machinery is all enclosed in a frame building 28 x 94 ft. The concrete materials are brought into the house at the rear of the post machine and storage room is provided in the house for a considerable quantity of material so that it may be warmed for use in cold weather. The drawing shows piles of stone chips and sand. The chips or screenings were tried experimentally because they could be obtained at practically no cost. They were not satisfactory, however, as the concrete seemed to lack strength and the posts were not smooth. A mixture of sand and screenings was tried with somewhat better results.

A supply of small gravel containing a large proportion of sand was then obtained and this material is superior in every way to the others mentioned. It can be obtained in any desired quantity at a moderate price and will undoubtedly form the standard material for this work.

As the posts are taken out of the machine they are placed on a push car standing on the track shown at the upper part of the drawing. In cold weather they are stored in the house for a few days before being taken out of doors. In warm weather they

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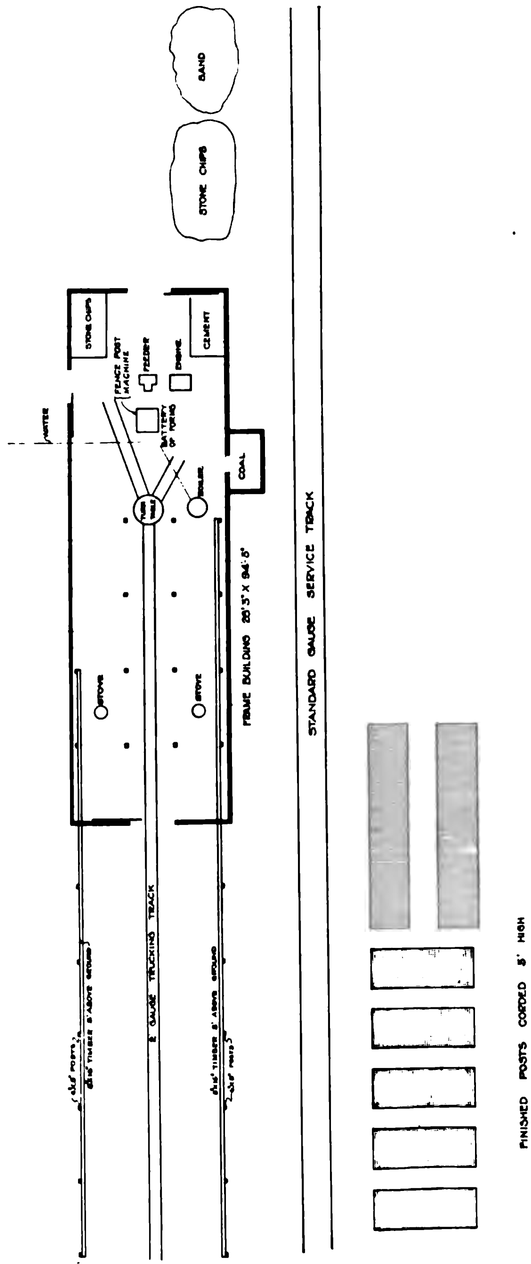

FIG. 1.—LAYOUT OF REINFORCED CONCRETE FENCE POST PLANT, HAVELOCK, NEB.

are stored indoors or out as may be most convenient. They are removed from the molds a day or two after being made and stored against the heavy timber racks until ready for shipment or until they are strong enough to be corded up in piles. For a week after being removed from the molds they are thoroughly wet down once a day and in summer protected from the sun by tarpaulins.

Fig. 2 shows the post machine. In the rear is the measuring machine consisting of two hoppers for gravel and cement. A

FIG. 2.—CONCRETE FENCE POST MACHINE.

small conveyor operates in the bottom of the hoppers and dumps the materials in proper proportions into a boot. Here it is picked up by a bucket elevator and taken to the mixer at the top of the machine.

The mixer is a large shallow bowl with a concave bottom. A number of paddles rotate in this bowl and mix the concrete, water being sprayed on it from a perforated pipe. In the bottom of the mixer is a hole which is closed by a form of gate valve, and through which concrete is discharged into the molds below.

Under the mixer is a turntable arrangement which holds four molds at a time. There is also a jolting device so arranged that as each mold is being filled it is alternately raised and dropped through a distance of perhaps an inch. This tamps the concrete effectually and ensures a smooth finish.

Two forms of reinforcement are used. The one which was used last year is made from sheets of No. 24 or No. 26 plain black iron. The sheets are passed through a machine which cuts out half the reinforcement for a post at each pass. At the same time long slits are cut in the iron and the edges of the slits turned up. Two strips of this material are inserted in the mold to form the reinforcement for one post. It has not been entirely satisfactory, however. If the concrete material is a bit too coarse it does not run through the slits readily and the posts are not well filled out. The reinforcement also has a tendency to get out of place during filling and is occasionally found to be near the center of the post instead of at the outsde.

Fig. 3 shows an improved style of reinforcement now coming into use. It is made entirely of wire, each wire being crimped to insure a bond with the concrete. The material is shipped knocked down, as shown at the left of the photograph, and is quickly made up into the cages shown at the center. The wires may be of such size as needed to give the required strength and the concrete flows around it without obstruction.

Fig. 4 illustrates the post and also the method of fastening the wires to the posts. By reference to the cross sections it will be seen that there is a groove in one side of the post and that the holes through the post are offset, being smaller on the grooved side of the post. These holes are of such a size that a ten-penny nail can be pushed through them until the head brings up against the offset. The fence wire is then placed on top of the projecting end of the nail and the latter bent up around the wire until its point is curled back into the groove, thus holding the wire tight against the post. This work is done with a small tool which is shown in successive positions in the upper part of the drawing. This is a very cheap and effective fastening. One difficulty has developed in connection with it, however. The pins with which the holes through the post are made are necessarily all alike. The post is tapered. As a result the small part of the hole through

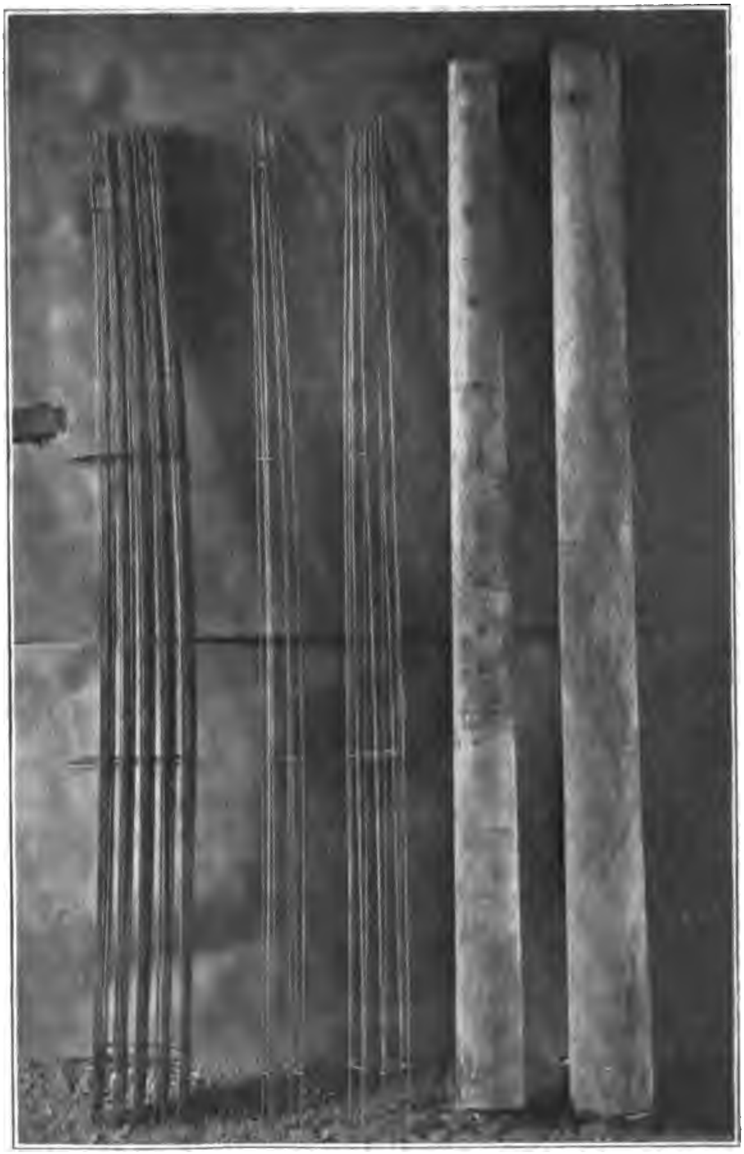

FIG. 3.—WIRE REINFORCEMENT AND FINISHED POLES.

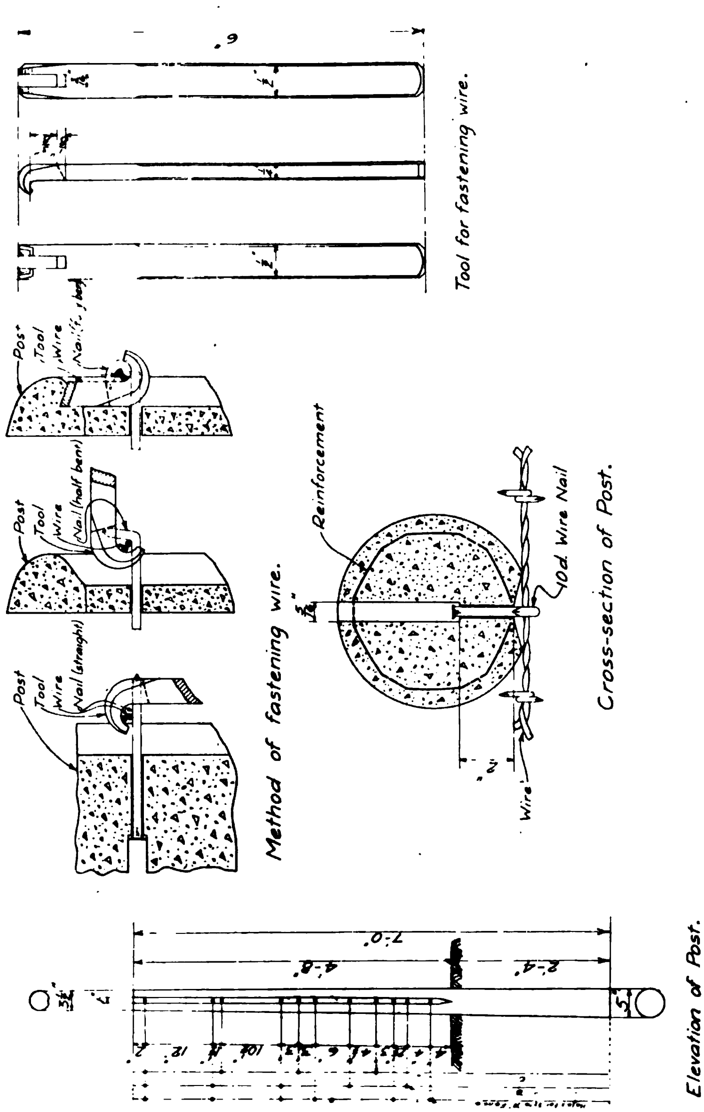

FIG. 4.—CONCRETE FENCE POST AND METHOD OF FASTENING WIRES.

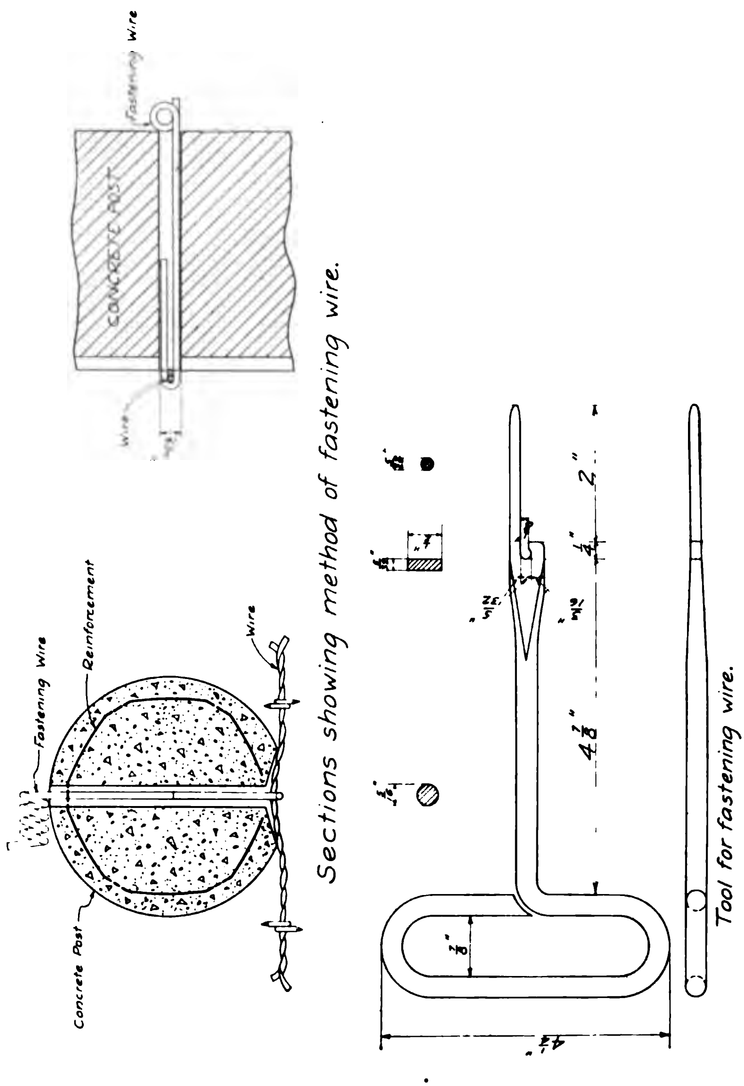

FIG. 5.—METHOD OF FASTENING WIRES ON CONCRETE FENCE POSTS.

which the shank of the nail passes is longer near the bottom of the post than at the top. Consequently the nails at the top stick out too far and when bent around the fence wire the points strike the bottom of the groove and make it difficult to pull the wire up close to the post. A tapered strip is now attached to the outside of the mold under the heads of the pins. It is of such a thickness that the offset in the holes is a uniform distance from the grooved side of the post and all nails project the correct distance to insure proper fastening of the wire.

Another method of fastening the wires is shown in Fig. 5. It will be seen that the hole through the post is of uniform diameter and a piece of wire with one end doubled back is substituted for the nail. The other end of the wire projects at the back of the post and by means of the tool shown this is twisted up into a corkscrew. The fastening has not yet been tried out in practice but its use on an experimental post seems to indicate that it is simple and efficient. It was designed for use with a round post having no groove.

The machine will turn out 400 posts per 10-hour day. As previously stated, it is not yet known how low the cost can be gotten but it can be made such as to compete with cedar posts.

It must not be expected that a concrete post will stand the degree of rough handling which can be given a wooden post. It is easy to make them strong enough to withstand the loads which come upon them in a fence and they are in no sense fragile. But some degree of care must be used in handling them or cracked posts will result. If concrete posts are used with the same care and intelligence required in any other form of permanent construction satisfactory results may be obtained.

The annual consumption of cedar posts on a large railroad runs into hundreds of thousands. Rot and grass fires eat them up, unruly steers break them off and the ever-present hobo finds them good fuel for his campfire, provided he can get them without too much labor. None of these agencies except the steer have any effect upon concrete posts and there is a large field for their use. They must be produced cheaply and in large quantities, however, and must be strong enough to stand rough handling as well as to resist the forces which come upon them in service.

DISCUSSION.

The President. PRESIDENT HUMPHREY.—Do you find any saving in maintenance?

Mr. Hotchkiss. MR. L. J. HOTCHKISS.—I have no doubt there is a material saving in maintenance, but we have not used the posts long enough to know what it amounts to.

There is one other desirable feature of concrete posts which I did not mention and that is their appearance. A man accustomed to the ordinary cedar post will be greatly surprised when he first sees a fence built with concrete posts. Being of uniform color, size and shape they make a very attractive fence and add much to the appearance of a field or a railroad right-of-way.

Mr. Arp. MR. C. K. ARP.—I would like to ask Mr. Hotchkiss how he determines what weight of post and the percentage or amount of reinforcement to use in a post and also how to place it.

Mr. Hotchkiss. MR. HOTCHKISS.—The amount of reinforcement has so far been decided by experiment and experience. The manufacturers of the machine had made a series of experiments which convinced us that their post would stand ordinary use at least. Experience, however, shows a larger per cent of breakage in handling than desirable and we are now changing the reinforcement from 5 to 6 longitudinal No. 8 wires per post, which is 4 in. diameter at the top and 5 in. at the other end, 7 ft. long. It is easily possible by increasing the size of the wires, the number, and the size of the post if necessary, to get any strength that would ordinarily be required. There is no limit to what you may do in that direction if you are willing to pay for the material and labor necessary to get the strength desired.

Mr. Meade. MR. JOHN M. MEADE.—I would like to ask Mr. Hotchkiss as to the proportion of concrete, if it is the same as in ordinary bridge work.

Mr. Hotchkiss. MR. HOTCHKISS.—It is about the same as used in reinforced work, 1 part of cement to 4 parts of gravel containing sand. This is equivalent to what is ordinarily known as a 1 : 2 : 4 : mixture. This gravel comes from the river bed and contains a great deal

more sand than would usually be used in concrete work. In this class of construction it is necessary to have an excess of mortar in order to get the concrete placed in the long cylindrical molds without voids. Mr. Hotchkiss.

MR. C. W. BOYNTON.—I believe Mr. Hotchkiss said that he expected to cut the price down as to cost. I would like to know about what the posts cost at present. Mr. Boynton.

MR. HOTCHKISS.—I don't know, Mr. Boynton, what they are costing. We have no accurate figures as yet. I have been told by other parties who are making these posts that they are doing it for 15 ct. per post. I think they can be made for not to exceed 20 ct., and I hope to get it down to 15 ct. I have been told that another railroad company which has been in the business longer than we have is making them for 12 ct. These figures are hearsay and I can not vouch for their accuracy. Mr. Hotchkiss.

MR. BOYNTON.—You consider the price of the posts then reasonable; that is, it would be preferable to a cedar post at present? Mr. Boynton.

MR. HOTCHKISS.—By all means. I do not recall the exact cost of cedar posts, I think it is around 12 or 13 ct. a piece. The life of a cedar post, I suppose, would be anywhere from 2 to 8 years, depending on what happens to it. If it does not catch fire from grass fires it may last 6 to 10 years. With a concrete post, which will certainly last 10 years, at 15 ct. against a cedar post, which may burn at any time, at 12 or 13 ct., there is surely economy in the use of concrete posts. Mr. Hotchkiss.

MR. BOYNTON.—I am afraid some might get a wrong impression from that remark of Mr. Hotchkiss that the concrete post will last as long as 10 years. I presume there is an average life to a post. Some might be broken accidentally. But barring the accidental break the post would last indefinitely. So I think that we can well afford to figure that the average life of a line of posts would be very much greater than 10 years. Mr. Boynton.

MR. HOTCHKISS.—That will depend somewhat upon the location of the fence. In railroad work, especially in the West, many changes in location and alignment are being made and additional tracks built. The moving of right of way fences incident to such operations will result in many broken posts, whatever the material of which they consist. Where a fence of concrete Mr. Hotchkiss.

Mr. Hotchkiss. posts is undisturbed, however, I should not put any limit on the life of the posts.

Mr. Van Hook. Mr. J. M. Van Hook.—In regard to the cost and serviceability of concrete fence posts, I would say that in Idaho with cement at $2.80 a barrel and sand and gravel about 30 ct. a cu. yd., we are making posts at 16 ct. each. The post is in cross-section an equilateral triangle, the sides being 3½ in. at the top and 6½ in. at the bottom, 6½ ft. long, reinforced with three ¼-in. rods. The wood post can be obtained for 20 ct., whereas 30 ct. is being paid for concrete posts. The line posts are sunk about 2 ft. in the ground, but the corner posts 3½ ft. In setting the latter a hole about 2 ft. square is prepared, the post set in and the hole filled up with concrete, no braces on the corner posts being used. The posts weigh about 72 lb. each and their weight is sufficient to prevent vibration by wind and they do not become loose like wooden posts.

The best post is made with wet concrete and is not as liable to breakage as one made of dry concrete. On one job 700 posts were hauled over eight miles of sage brush road in Idaho and only one broke. Ninety-five per cent. of the failures of concrete posts has been due to the use of dry concrete.

Mr. Arp. Mr. Arp.—A study of the fence post proposition has shown that the failures are due principally to the method of manufacture and to sudden direct shocks or jars to the post in place, rather than the steady pull or strain caused by anything running against the line wires. Many fence posts are not properly handled after leaving the molds, the rough handling causing minute cracks, not appearing immediately, but after the posts are in place moisture reaches the reinforcement with resultant failure of the post. It has been found that the post should be made of such cross-section that there will be no small sections, so that abrasion from wheel hubs or wagons, etc., will not knock or spall off small sections of concrete and thereby expose the reinforcement. If there can be determined the proper cross-section and reinforcement to take care of the strain and resist shocks, a post will be secured to meet all requirements.

Mr. Dougherty. Mr. G. A. Dougherty.—Experiments have shown that it is impossible to make a plain concrete post in the smaller sizes as strong as a wooden post of the same size. By the use of reinforcement this can be done, however. In order to properly protect the

reinforcement, which is the life of the post, it is necessary to increase the size of the post by using more concrete. A post can of course be designed for a certain stress the same as any other reinforced concrete member. It is cheaper to use more concrete than to resort to hooped reinforcement when the plain rods are not sufficient. In order to obtain the maximum protection of the reinforcement it is necessary to use wet concrete. If dry concrete is used, the reinforcement must be galvanized so that the moisture coming through the porous concrete will not cause it to rust and crack the concrete. The difference between wet and dry concrete is that the forms can immediately be taken from the latter, whereas the former must remain in the forms until the concrete has hardened. **Mr. Dougherty.**

Local conditions will determine the best and most economical aggregates to be used, the proportions of the concrete depending upon the large aggregate or gravel. With the average aggregate the usual proportion is 1 part cement to 4 parts of gravel, $\frac{3}{8}$ in. in size down to sand. The strain in a fence line is supposed to be taken by the anchor or corner posts which are of greater cross-section, and larger aggregates are used.

It is not necessary to have the concrete post as strong at the beginning as a wooden post because the latter continually deteriorates, whereas the former gains in strength. The loss from breakage can well be considered counterbalanced by this saving.

The molds should be of simple construction, preferably of metal, as wood soon checks and warps out of shape. Again wood absorbs moisture from the mass, whereas metal will not and at the same time gives a smoother surface to the post.

The reinforcement used should be of such size and character as can be readily obtained in small quantities. The method of fastening the wire to the post should be as simple as possible and the place of attachment variable in the case of posts to be sold from stock.

In regard to the method of curing, this is either by sprinkling or by steam. In the first case the posts should of course be well protected so as not to dry out too fast.

MR. ARP.—The manufacturers of post molds have been advocating and claiming that a post can be made for from 9 to 11 ct., which is wrong. Taking, for instance, a post 3 in. square at the top **Mr. Arp.**

Mr. Arp. and 5 in. at the bottom, reinforced with $\frac{3}{4}$ in. rods, with cement at \$1.50 a barrel and sand and gravel at about \$1.00 a cu. yd., the materials alone for a post, without labor, will cost about 20 ct. It should not be expected that a good concrete post can be bought on the market for the same price as a cedar post in the lumber district.

Mr. Dougherty. MR. DOUGHERTY.—Of course the cost of a concrete post will depend upon local conditions entirely, the average cost of the materials being about 20 ct.

ANNUAL REPORT OF THE EXECUTIVE BOARD.

The Executive Board presents herewith the following report on the condition of the affairs of the Association.

Meetings.—Since the last Convention, the Elective Members of the Executive Board have held one meeting, the Executive Board has held four meetings and the Executive Committee has met several times. In addition many conferences were necessary to carry on the work of the Association. An abstract of the minutes of these meetings is appended hereto.

Membership.—The membership at the close of the last meeting was 1,048, since that time 181 new members have been added, 26 members have resigned, 2 members have died and 109 have been dropped for the non-payment of dues. At the present time the membership is 1,092.

Report of the Treasurer.—

RECEIPTS.

Balance, July 1, 1910		$794.67
Dues	$4,080.00	
Dues, Contributing Members	639.00	
Sale of Publications	3,683.03	
Exhibition, New York	1,000.00	
Membership Certificate	134.50	
Membership Imprint	8.00	
Subscriptions	110.00	
Convention	80.25	
Interest on Deposits	21.38	
Miscellaneous	14.38	
Total Receipts		9,770.54
		$10,565.21

EXPENDITURES.

Salaries	$2,582.50
Office Rent	585.00
Office Equipment	182.85
Postage, Printing, Stationery, etc.	1,001.87
Sectional Committee Expense	17.40
Meetings of Executive Board	115.15
Publications	1,965.65

Convention, Chicago	$326.10	
Convention, New York	2,247.35	
Membership Campaign	327.94	
Membership Certificates	247.56	
Membership Pins	59.65	
Subscriptions	19.40	
Miscellaneous	34.40	
Total Expenditures	$9,712.82	
Balace, June 30, 1911	852.39	
		$10,565.21

NOTE.—Due John C. Winston Co. for bills incurred and unpaid during year ended June 30, 1911, for printing Proceedings, etc., $2,632.70.

PHILADELPHIA, 2d August, 1911.

We hereby certify that we have audited the accounts of the Treasurer of the National Association of Cement Users for the year ended June 30, 1911, and found that the dues collected and receipts from sales of publications, etc., as they appear in the cash book have been properly accounted for, and that all disbursements were substantiated by vouchers.

The foregoing report of the Treasurer is in agreement with the books of account.

(Signed) LYBRAND ROSS BROS. & MONTGOMERY,
Certified Public Accountants.

Proceedings.—The Proceedings of the Association have increased in size from year to year, reaching a maximum in Vol. VII of 920 pages. By reason of this increased size, it was found impossible to substantially bind this volume in paper covers and it was necessary therefore to bind in cloth; your Board felt justified in so doing and asking the members to voluntarily pay the increased cost of 50 ct., the regular charge for cloth binding. These Proceedings are one of the principal assets of the Association and this volume has, if anything, raised the standard set by previous volumes. This fact is testified to by the increased sale to technical societies and libraries as well as to those who are non-members of the Association.

This increase in the size of the Proceedings has naturally led to an increased cost of publication. The Board believes that an increased charge for advertising is justified and should result in sufficient revenue to defray the cost, which is not the case at the

present time, making the Proceedings a source of annual deficit. The Board feels strongly that these Proceedings should be self-supporting, if not a source of considerable revenue. It believes further that it should be possible to secure a much wider distribution through sale which will result in a very material increase in our revenue. It recommends to the incoming Board that a serious effort be made to very materially increase the revenue from advertising and sale of Proceedings.

An effort is made each year to select the authors of papers so that the program as a whole will contribute information on subjects not covered by previous Proceedings and thus embrace the various uses of cement. This so far has resulted in a collection of information in our Proceedings which make them now in the nature of a valuable reference book and the Board is of the opinion that this practice should be continued so that the Proceedings will be a valuable adjunct in the offices of users of cement as a book of reference.

The Board also takes pleasure in bringing to the attention of the Association the fact that in many matters, especially those relating to the design of flat slabs, there is to be found in the Proceedings, information of the greatest practical value that cannot be obtained from any other source, and many instances have come to the attention of the Association which has made this sufficient reason for membership.

Recommended Practice and Standard Specification.—It is apparent to your Board that the Recommended Practice and Standard Specifications of this Association are becoming more and more recognized and are exerting a constantly increasing influence. The work which the Association is doing in this direction is still further exemplified in the many excellent recommendations made by the standing committees of the Association.

It should be a matter of the greatest gratification to the members to know that the foreign concrete associations appreciate and follow to a very large extent all the standards and recommended practice which have been formulated by this Association. And in this country it should be a further matter of gratification to know that our building regulations are either bodily incorporated or have been used as a basis in the framing of regulations in many of the largest cities in this country.

Convention.—The Board believes its feeling that the Cement Show is a desirable adjunct to our Convention, is shared by a large majority of the members and it has therefore deemed it advantageous to continue its affiliation with the Cement Products Exhibition Company. This company has always manifested a keen active interest in the welfare of the Association, has contributed largely in a financial way and the Board feels that this co-operation has been very helpful to the Association and that it has been beneficial to the Exposition Company. It is certain that this affiliation has relieved the Association of an enormous amount of work which formerly resulted from the annual cement exhibition.

The Board reiterates its opinion expressed last year that these expositions have so increased as to make them undertakings of no inconsiderable magnitude and for the success of which there has thus far been required months of careful preparation. It is certainly apparent to your Board that the liabilities involved in managing a show of this character are such that the Association would not be justified in undertaking the management on the present scale.

Income—The Executive Board feels that but few members, and certainly a very small part of the industry at large, appreciate the conditions under which the Association performs its functions and how much it has been handicapped through lack of funds. While the Board appreciates the fact that the Association is growing in importance and its work is commanding constantly increasing support, yet it feels compelled, in justice to itself, to bring to the attention of the Association, at this time, the utter insufficiency of the support afforded it by interests which should be glad to place it in a position where it could enjoy a still greater field of usefulness.

The amendment to the By-Laws, which is submitted for consideration at this Convention, providing for an increase in dues of $5.00 per annum, has been carefully considered by the Board. The canvass of the letter ballot, called at the request of the Board, has been carefully analyzed and it is the unanimous opinion of the Board that this increase in the annual dues of the members is inexpedient at this time, especially so since the Board does not believe that this in itself will give the additional revenue necessary to provide for the present annual deficit. The Associa-

tion being entirely for educational purposes, in advancing the proper use of cement, it appears desirable that the annual dues should be such as not to be a consideration in determining the value of membership. The Board believes that the work of the Association can be very materially propagated by a much wider distribution of the information contained in the Proceedings and it is decidedly of the opinion that this can be best accomplished by a greatly increased membership. Your Board feels that the cement industry should have a sufficient appreciation of the value of the work of this Association to be willing to contribute, in the shape of contributing or sustaining memberships or voluntary contributions, sufficient funds to make the Association self-supporting, and further believes that with such a support for the next few years, the Association will reach a position where its own revenue will be sufficient to make it self-supporting. The Board is unwilling to assume the responsibility for the continued existence of this Association without adequate co-operative support from those whom it believes are materially benefited by its existence. The Board, therefore, in recommending that the dues be not increased is confident that there will be such a response from those interested in the welfare of the Association that its financial affairs will be at once remedied and placed in a healthy condition.

At the Buffalo Convention in 1908, the Convention instructed the Executive Board to establish a permanent headquarters and to employ a permanent Secretary, in order that the efficiency of the Association might be increased. It was proposed at that time to raise, by contributing memberships, an annual guaranteed fund of $2500, to cover this increased expense. The maximum amount raised by the contributing membership committee was less than $800 and this has therefore resulted in an annual deficit of upwards the difference between this latter amount and that promised from contributing membership, averaging perhaps $2,000 per annum. Prior to the establishment of headquarters the finances of the Association showed an annual surplus.

The Executive Board is decidedly of the opinion that to sufficiently curtail its expenses to keep within its revenue, would mean an abandonment of headquarters and permanent Secretary, which would be a retrogression that would be extremely hurtful to the future of the Association. It is the opinion of the Board that

the industry will not only provide sufficient money through donations and contributing membership, but that each member should constitute himself a committee of one to contribute or secure contributions of whatever size may lay within his power and thus show his co-operation in the work of the Association.

The present volume of Proceedings remains unpaid for and there is owing an account of last year's Proceedings, which in the aggregate will represent the deficit at the close of the fiscal year amounting to about $4,000, and it is this deficit that the Executive Board feels should be raised immediately in the manner above indicated.

Amendments to By-Laws.—The Executive Board recommends certain amendments in the By-Laws, which are the result of experience with our present By-Laws, and feels the Association as reörganized will be in a much better and much more businesslike condition and we hope the amendments as proposed will be unanimously adopted.

Research Fund.—Many committees of the Association have found in the course of their work the necessity for information on practical subjects which was either not available or could not be obtained. The need of such information has been particularly apparent in the work of the Committee on Reinforced Concrete and Building Laws, and in response to the request of this Committee the President and Treasurer of the Association raised, through voluntary contributions, money for the purpose of conducting specific tests under the direction of this Committee. The valuable results of this work are reflected in the report of the Committee on Reinforced Concrete and Building Laws this year. It was the feeling of the Board that money raised in this way should be handled by the Association and not by the Committee or individual officers of the Association, and should be kept distinct from the regular funds of the Association. It was deemed, thereford, expedient to create a research fund and the following is a statement of the receipts and expenditure of the money thus raised.

Contributions	$1,850.00
Ferro-Concrete Construction Company.	
Turner Construction Company.	
Aberthaw Construction Company.	

Salmond Brothers Company.
American Concrete Steel Company.
Tucker and Venton.
Industrial Engineering Company.
Turner Forman Concrete Steel Company.
Expanded Metal Engineering Company.
O. W. Shelly.
Wm. Steele and Sons Company.
Moyer Engineering and Construction Company.

Expenditures:		
Wenalden Test	$710.83	
Turner Carter Test	781.50	
		$ 1,492.33
Balance		$357.67

It should be noted, however, that the amount of money raised represents the funds which actually passed through the Association, but does not represent the total money necessary for carrying on these tests. In addition to the expenditures noted, which are conservatively stated as upwards of $8,000, under an arrangement by the President with Professor A. N. Talbot of the Engineering Experiment Station of the University of Illinois, the latter agreed to furnish facilities for the details of conducting the tests and for collating the results without charge other than railroad and hotel expenses incident to the test, with the understanding that the Engineering Experiment Station of the University of Illinois should have the right to publish the data thus obtained after it had been presented to and published by this Association, and provided further that due credit be given to the Association for the use of this data in whatever publication it might appear. It will thus be seen that the investigations covered in the report of your Committee on Reinforced Concrete and Building Laws represents an aggregate expenditure of upwards of $10,000. The Board feels that the Association owes much to those who have contributed not only cash but the co-operation of the Engineering Experiment Station of the University of Illinois and others without which these tests would not have been possible.

APPENDIX.—MINUTES OF MEETINGS OF THE EXECUTIVE BOARD.

MEETING OF THE ELECTIVE MEMBERS OF THE EXECUTIVE BOARD, HELD DECEMBER 19, 1910, AT THE PRINCE GEORGE HOTEL, NEW YORK, N. Y.

Present: Richard L. Humphrey, President; Edward D. Boyer, First Vice-President; E. S. Larned, Third Vice-President; and Ira H. Woolson, Fourth Vice-President.

On motion, Mr. H. C. Turner was elected Treasurer and Mr. Edward E. Krauss, Secretary for the ensuing year.

On motion, Chairmen of the Sectional Committees were appointed as follows:

Building Blocks and Cement Products.—Percy S. Hudson.
Exhibition.—H. S. Doyle.
Fireproofing.—Rudolph P. Miller.
Insurance.—William H. Ham.
Reinforced Concrete and Building Laws.—A. E. Lindau.
Roadways, Sidewalks and Floors.—C. W. Boynton.
Treatment of Concrete Surfaces.— Leonard C. Wason.

MEETING OF THE EXECUTIVE BOARD, HELD DECEMBER 20, 1910, AT THE PRINCE GEORGE HOTEL, NEW YORK, N. Y.

Present: Richard L. Humphrey, President; Edward D. Boyer, First Vice-President; E. S. Larned, Third Vice-President; Ira H. Woolson, Fourth Vice-President; Rudolph P. Miller, Sectional Vice-President; H. C. Turner, Treasurer; and Edward E. Krauss, Secretary.

The applications for membership received during the Convention were approved.

It was the sense of the Board that all journals receiving the privilege of publication of the papers presented before the Convention should be contributing members.

It was decided to appoint a Committee on Advertising to assist the President in obtaining sufficient advertisements to permit of the early publication of the volume of Proceedings.

On motion, a committee to obtain advertisements at the New York Cement Show was appointed as follows: J. P. H. Perry, Thomas M. Vinton and Edward D. Boyer.

According to the By-Laws the Executive Committee shall consist of the President, Secretary, Treasurer and two members of the Executive Board. On motion, Edward D. Boyer and Ira H. Woolson were appointed as the two additional members of the Executive Committee.

The time and place for the Eighth Annual Convention were referred to the Executive Committee with power to act.

On motion, it was decided that three copies of the Proceedings of the New York Convention be sent to the U S. Military Academy at West Point and three copies to the U. S. Naval Academy at Annapolis. The President was authorized to present a copy of the Proceedings to the Foreign Delegates and the American Official Delegates.

On motion, the President was instructed to advise the Bankers' Trust Company that hereafter the checks of the Association would be drawn by Edward E. Krauss as Secretary, instead of as Acting Secretary.

Adjournment.

MEETING OF THE EXECUTIVE BOARD, HELD AUGUST 2, 1911, AT THE HOUSE OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS, NEW YORK, N. Y.

Present: Richard L. Humphrey, President; Edward D. Boyer, First Vice-President; Ira H. Woolson, Fourth Vice-President; C. W. Boynton and Rudolph P. Miller, Sectional Vice-Presidents; and Edward E. Krauss, Secretary.

The minutes of the meetings of the Executive Board held on December 12–13, 1910, and of the Elective Members of the Executive Board, December 19, 1910, and of the Executive Board, December 20, 1910, were approved as read.

On the absence of the Treasurer, owing to illness, the Secretary presented the statement of cash receipts and expenditures for the year ended June 30, 1911, as approved by the Certified Public Accountants. The report was accepted and ordered filed.

The Secretary reported the total membership on June 30, 1911, as 1149, and that 26 members had resigned, 2 died, 14 were out of business and 95 were dropped for non-payment of dues; a total loss of 137, making the net membership 1012.

It was decided that the three papers presented by the Hungarian Society of Engineers and Architects at the Seventh Annual Convention, be translated into English at an estimated cost of $125.00.

The President presented a letter and bill from the Trunk Line Association stating that inasmuch as 1000 certificates and round-trip tickets had not been presented for validation at the time of our convention, the reduced railroad rate of a fare and one-half would not be effective and that the Association was indebted to the Trunk Line Association for the difference between the full fare and the one-half return fare on such certificates as were used, amounting to $2081.

It was the sense of the Board that the President advise the Trunk Line Association that the matter had been carefully considered by the Executive Board and from the facts in the case were of the opinion that the agreement had been complied with and that the Association was under no obligation to pay their bill of March 11, 1911, $2081.

The President reported that Volume VII of the Proceedings had not been printed owing to the receipt of an insufficient number of advertisements.

It was the opinion of the Board that the printing of the Proceedings should not be delayed more than ten days even if sufficient advertisements were lacking. It was decided to print 2000 copies.

In view of the rapid development of the cement industry it was deemed that the subject-matter of the annual volumes would become obsolete in five to ten years and it was, therefore, decided not to electrotype the pages of the Proceedings.

In view of the low stock of Volume III of the Proceedings it was decided that hereafter when a stock of volumes reached seventy-five copies, such volumes be only sold in complete sets and not as individual copies.

A letter from Mr. E. L. Corthell was presented inquiring as to the terms under which the Corthell Library at Brown University, endowed by himself, could obtain the Proceedings after his death. In view of the special circumstances in the case it was to suggest to Mr. Corthell that he continue membership until his death, and that after that the Corthell Library would be furnished the Proceedings at the same price made to members for the additional copies.

The question of exchange of publications with other organizations was considered and it was decided that the exchange list be submitted annually to the Executive Board, for approval.

In the matter of the request of the German Concrete Association for twenty copies of our Proceedings, it was decided that three copies be sent them, being as many as the Association could afford at this time.

In view of the courtesy of the American Society of Civil Engineers in furnishing accommodations in the Society House for the meetings of the Executive Board, it was decided that a copy of the Annual Proceedings be sent the American Society of Civil Engineers with the compliments of the Executive Board.

A letter was presented from the Canadian Cement and Concrete Association advising that at the last meeting of its Executive Committee, it was decided to present for the consideration of our Executive Board the suggestion that the President and Secretary of the Canadian Association become members of the National Association and that in exchange the President and one other representative of the National Association become members of the Canadian Association; such a condition probably leading to assistance in securing a better working together of the two organizations. In view of the fact that the Canadian Cement and Concrete Association is the representative organization of its kind in Canada, it was decided to suggest that the President in each organization hold membership in the other organization; also that in the Proceedings of the Canadian Association there appear a table of contents of the current volume of the National Association and vice versa.

The question of Contributing Membership was discussed and the President was instructed to write a letter to each contributing member thanking them for their cooperation in the past and stating it as the desire of the Executive Board to make the Association self-supporting as soon as possible

so as to relieve the contributing members, and that the Board hoped they would continue their contributing membership for the current year.

The President reported that the Committee on Concrete and Reinforced Concrete had made arrangements for special tests of concrete structures in various parts of the country, and that contributions to cover the expenses had been promised. It was decided to open an account in the books of the Association to be termed "Research Fund," and that all subscriptions obtained by committees for special work be paid into the treasury of the Association and disbursed on regular vouchers.

In the matter of tests of buildings under the direction of the Committee on Concrete and Reinforced Concrete, it was decided that if Professor Talbot furnished free of charge the services of himself and assistants in making the tests and compiling the data, the traveling and hotel expenses would be paid by the Association, with the understanding that the results are first to be published by the Association with a note that the work had been done in cooperation with the Engineering Experiment Station of the University of Illinois, and that the results could then be published by Professor Talbot as a bulletin of the Engineering Experiment Station of the University of Illinois, provided that full credit be given to the fact that these tests were made under the auspices of the National Association of Cement Users.

It was decided that the special Committee on Education be considered a regular committee and its Chairman be a Sectional Vice-President of the Association.

The present organization of the Association was discussed and the President requested at the next meeting of the Board to present an outline for a reorganization.

The President reported his negotiations with the Cement Products Exhibition Company in regard to the Eighth Annual Convention, together with the results of his canvass of the business men of Kansas City and it was decided to refer the matter to the President with power to act.

A letter from Mr. Robert F. Havlik in regard to the time for the first sessions of the Convention as conflicting with the work of the exhibitors was referred to the President.

The Secretary, in the absence of the Treasurer, presented a statement of the income and expenses applying to the separate fiscal years beginning January 1, 1908. The financial affairs of the Association were discussed, and it was decided to take up at the next meeting of the Board the question of increasing the dues so as to place the Association on a self-supporting basis. The Treasurer presented, by letter, the suggestion that a committee, consisting of E. D. Boyer, E. S. Larned and H. C. Turner, be appointed to consider and report a definite plan for financing the Association. In view of the discussion, action on this suggestion was deferred until the next meeting.

The bill of The John C. Winston Company for printing of Vol. VI of the Proceedings was considered and, as requested, the Secretary was instructed to draw a voucher for $1000 on account.

Adjournment.

MEETING OF THE EXECUTIVE COMMITTEE, HELD NOVEMBER 21, 1911, AT 11 BROADWAY, NEW YORK, N. Y.

Present: Edward D. Boyer, First Vice-President; Ira H. Woolson, Fourth Vice-President; Henry C. Turner, Treasurer; and Edward E. Krauss, Secretary.

The Secretary reported Vol. VII of the Proceedings contained 920 pages and that the cloth and leather bound copies had been distributed. Practical difficulty in properly binding such a large volume in paper covers had developed and the Secretary suggested that a sufficient number of volumes be bound in cloth to supply the members entitled to paper bound copies; that the remainder be held unbound until needed. Several volumes in paper binding were exhibited demonstrating the poor results obtained.

It was decided to send to those members entitled to paper binding, a copy of Vol. VII in cloth binding, and to address such members by letter, calling attention to the circumstances and enclosing bill for fifty cents to cover the cloth binding, asking them, in view of the limited finances of the Association, to assist in defraying this additional expense.

Adjournment.

MEETING OF THE EXECUTIVE COMMITTEE, HELD FEBRUARY 29, 1912, AT 11 BROADWAY, NEW YORK, N. Y.

Present: Richard L. Humphrey, President; Edward D. Boyer, First Vice-President; Ira H. Woolson, Fourth Vice-President; Henry C. Turner, Treasurer; Edward E. Krauss, Secretary; and on invitation, Rudolph P. Miller, Sectional Vice-President.

The Treasurer reviewed the finances of the Association which were fully discussed.

The Revisions of the By-Laws, as sent to the members in Circular No. 30, were discussed, and on motion it was decided to send to the members a letter with a reply postal card requesting their opinion as to increasing the annual dues.

Adjournment.

MEETING OF THE EXECUTIVE BOARD, HELD MARCH 11, 1912, AT HOTEL BALTIMORE, KANSAS CITY, MO.

Present: Richard L. Humphrey, President; Edward D. Boyer, First Vice-President; Edward E. Krauss, Secretary; E. J. Moore, representing H. C. Turner, Treasurer; Rudolph P. Miller and C. W. Boynton, Sectional Vice-Presidents; and P. Austin Tomes and C. A. Chapman, on invitation.

The Secretary reported that since the last Convention 151 new members had been elected and that 109 members were dropped for the non-payment of dues, 26 members resigned and 2 members were lost by death, making a total loss of 137. The total membership on March 9, 1912, was 1062.

The Secretary presented the following compilation of the reply postal cards sent out by the Executive Committee on February 29, 1912, requesting the members to advise whether they approve increasing the dues to $10.00 a year.

Favorable	177
Unfavorable	149
Total number of replies	326

The revision of the By-Laws and the finances were considered and it was decided to recommend that the dues remain at $5.00 a year. The President was instructed to prepare the annual report of the Executive Board.

In pursuance of the By-Laws the following committees were appointed:

Committee on Nomination of Officers:

E. J. Moore, Chairman, New York, N. Y.
D. A. Abrams, Ames, Iowa.
John E. Conzelman, St. Louis, Mo.
B. F. Lippold, Chicago, Ill.
L. T. Sunderland, Kansas City, Mo.

Committee on Resolutions:

Rudolph P. Miller, Chairman, New York, N. Y.
P. H. Bates, Pittsburgh, Pa.
Allen Brett, New York, N. Y.
F. L. Williamson, Kansas City, Mo.
Percy H. Wilson, Philadelphia, Pa.

MEETING OF THE EXECUTIVE BOARD, HELD MARCH 12, 1912, AT HOTEL BALTIMORE, KANSAS CITY, MO.

Present: Richard L. Humphrey, President; Edward D. Boyer, First Vice-President; Edward E. Krauss, Secretary; E. J. Moore, representing H. C. Turner, Treasurer; L. C. Wason and Percy H. Wilson representing L. W. Page, Sectional Vice-Presidents; and E. E. Howard, Chairman of the Committee on Banquet.

The banquet to be held on Friday evening, March 15th, was discussed and it was decided to set the price at $3.00 per cover.

The report of the Committee on Insurance was read by the President and it was decided to present the same at the Convention by title and to instruct the Committee on Insurance to revise its report before publication.

The finances were discussed and the report of the Treasurer accepted.

The annual report of the Executive Board was approved as prepared by the President.

REGISTER OF ATTENDANCE.
EIGHTH CONVENTION.

Richard L. Humphrey, Philadelphia, Pa.
Edward D. Boyer, New York, N. Y.
Arthur N. Talbot, Urbana, Ill.
E. S. Larned, Boston, Mass.
Ira H. Woolson, New York, N. Y.
Edward E. Krauss, Philadelphia, Pa.
H. C. Turner, New York, N. Y.
P. S. Hudson, Louisville, Ky.
Logan Waller Page, Washington, D. C.
H. S. Doyle, Chicago, Ill.
R. P. Miller, New York, N. Y.
W. H. Ham, Boston, Mass.
Robert A. Cummings, Pittsburgh, Pa.
Peter Gillespie, Toronto, Canada.
A. E. Lindau, Chicago, Ill.
C. W. Boynton, Chicago, Ill.
Sanford E. Thompson, Newton Highlands, Mass.
L. C. Wason, Boston, Mass.

Affleck, B. F., Chicago, Ill.
Allen, O. T., Chicago, Ill.
Anderson, W. P., Cincinnati, Ohio.
Arp, Clarence K., Chicago, Ill.
Ash, L. R., Kansas City, Mo.
Bates, P. H., Pittsburgh, Pa.
Bartholow, B. L., Neck, Mo.
Beck, J. P., Chicago, Ill.
Benedict, Ralph R., Kansas City, Mo.
Bickelhaupt, F. R., Cleveland, Ohio.
Brandt, J. W., Garnett, Kan.
Brown, Darius A., Kansas City, Mo.
Cement Age: Brett, Allen, Philadelphia, Pa.
Cement Era: Hanson, E. S., Chicago, Ill.

Chapman, C. M., New York, N. Y.
Chollar, W. T., Chicago, Ill.
Chubb, J. H., Chicago, Ill.
Clack, James N., Kansas City, Mo.
Collings, W. A., Kansas City, Mo.
Concrete: Boynton, Walter C., Detroit, Mich.
Craney, A. H., Kansas City, Mo.
Cross, Dr. Walter M., Kansas City, Mo.
Cunningham, A. O., St. Louis, Mo.
Darnell, James L., Kansas City, Mo.
Donnell, R. E., Kansas City, Mo.
Dougherty, G. H., Three Rivers, Mich.
Dreher, Frederick C., Denver, Col.
DuBois, C. R., St. Joseph, Mo.
Early, John, Newton, Kansas.
Egan, Louis H., Kansas City, Mo.
Eldridge, M. O., Washington, D. C.
Elmer, Bert E., Kansas City, Mo.
Engineering News: Wight, Frank C., New York, N. Y.
Engineering Record: DeBerard, W. W., Chicago, Ill.
Felt, J. H., Kansas City, Mo.
Forsee, George H., Kansas City, Mo.
Fox, S. Walters, Kansas City, Mo.
Fratt, F. W., Kansas City, Mo.
Gardner, Harry, Lawrence, Kan.
Gearhart, W. S., Manhattan, Kan.
Gibson, R. A., Hot Springs, Ark.
Gilbert, H. C., Kansas City, Mo.
Gordon, M. E., Chicago, Ill.
Green, Herbert P., Kansas City, Mo.
Groves, E. W., Ann Arbor, Mich.
Hall, R. F., Chicago, Ill.
Hanna, John V., Kansas City, Mo.
Harrington, J. L., Kansas City, Mo.
Heidenreich, E. Lee, Kansas City, Mo.
Howard, E. E., Kansas City, Mo.
Hoyt, Henry F., Kansas City, Mo.
Hudson, O., Kansas City, Mo.
Isenberg, Gerson, New York, N. Y.

Kinney, William M., Pittsburgh, Pa.
Knisely, Clyde J., Chicago, Ill.
Koch, H. C., Kansas City, Mo.
Leeson, R. V., Topeka, Kan.
Lehrack, Charles R., Lincoln, Neb.
Light, Charles P., Washington, D. C.
Luten, Daniel B., Indianapolis, Ind.
McGrew, Henry, Kansas City, Mo.
Marvin, Frank O., Lawrence, Kan.
Misner, F. H., Lincoln, Neb.
Moore, E. J., New York, N. Y.
Nicholson, George E., Kansas City, Mo.
Norcross, Hiram, Kansas City, Mo.
Northcott, B. H., Hesston, Kan.
Osborn, C. F., Howard, Kan.
Page, E. B., Oklahoma City, Okla.
Palmer, Peter, Oakland, Neb.
Paul, L. H., Des Moines, Ia.
Perry, J. P. H., New York, N. Y.
Powell, C. M., Chicago, Ill.
Prince, John, Kansas City, Mo.
Rock Products: Lippold, B. P., Chicago, Ill.
Root, Walter C., Kansas City, Mo.
Schreiber, H. V., Philadelphia, Pa.
Schultz, Edward H., Kansas City, Mo.
Scott, C. F., Oklahoma City, Okla.
Shainwald, Ralph L., Jr., New York, N. Y.
Sharp, William F., Moore, Mont.
Siedhoff, George, Kansas City, Mo.
Sims, Charles E., Worthington, Minn.
Slater, W. A., Urbana, Ill.
Sloane, C. B. Kansas City, Mo.
Slowski, August, Tonganoxie, Kan.
Smith, Charles A., Kansas City, Mo.
Snell, H. W., Chicago, Ill.
Snowhall, John T., Ellsworth, Kan.
Spahr, J. C., Skidmore, Mo.
Stapleton, E. R., Kansas City, Mo.
Stauffer, Abraham J., Joplin, Mo.

Steinmetz, A., Kansas City, Mo.
Stevens, A. H., Kansas City, Mo.
Stoddart, S., Kansas City, Mo.
Stone, O. O., Chanute, Kan.
Struckmann, H., Kansas City, Mo.
Sunderland, L. T., Kansas City, Mo.
Suter, H. A., St. Joseph, Mo.
Swenson, Godfrey, Kansas City, Mo.
Talbot, Kenneth H., Oak Park, Ill.
Taylor, J. C., Kansas City, Mo.
Tebbetts, George E., Kansas City, Mo.
Thayer, L. V., Minneapolis, Minn.
Thomas, Frank J., Oklahoma City, Okla.
Tomes, P. Austin, New York, N. Y.
Treadway, H. P., Kansas City, Mo.
Tritel, J. S., Kansas City, Mo.
Ubbink, Joe, Port Washington, Wis.
Warner, Charles D., Chicago, Ill.
Warner, John E., Benton, Mo.
Whipperman, Frank, Omaha, Neb.
Whited, Willis, Harrisburg, Pa.
Wig, R. J., Washington, D. C.
Williams, George A., Muskogee, Okla.
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